Earthquake Risk Assessment of Historical Structures

Dissertation

submitted to and approved by

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2007

*) Either the German or the Italian form of the title may be used.

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To my family, my parents and my sister, I apologize for those times I was absent or too concerned with my own work. Despite this, you always offered me support and rest.

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Cologne, April 2007 Marcel Urban

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Chapter 1

Introduction

1.1 Introduction

A commonly stated sentence is that the earthquake risk of historical structures is very high. This is assumed to be especially the case for tall and slender structures, such as churches. But now, what does this word *risk* really imply? Is *risk* similar to *vulnerability* or *reliability*? Is it a probabilistic or a deterministic concept? Are the consequences of structural damage to be considered and if, how might this best be done? Are a correct interpretation of the structural behaviour and a computation of the inherent risk possible or are the influences of uncertainties in the description of the process too large to provide reliable results in a practical way?

Numerous questions occur if a serious approach towards the topic of earthquake risk assessment of historical structures is planned. The new concept of risk assessments faces some rather old – but in some cases still unsolved – research problems. It is thus the main task to introduce and explain the concept of risk assessment and develop procedures for its application. Along the way a number of research tasks have to be addressed and solved which will be explained throughout each chapter. The written work presented in the following is structured according to the tasks to be performed in the risk assessment procedure which will be explained in chapter 2. It will lead step by step to the sample application explained in chapter 7.

1.2 Motivation

Historical buildings, especially churches and vaulted structures are very vulnerable with respect to ground motions such as earthquake shaking. Although it is sometimes alleged that these structures are already in existence for a very long time without having failed, it is mostly forgotten that they were rebuilt or enhanced in their structural performance. Some of them might simply never have experienced an extreme event but nevertheless suffer from ongoing material deterioration. Also, due to long return periods of extreme events the public is often not aware of the severe implications an earthquake might have. The importance of this matter is best described for Germany which is in general considered an area of small seismicity. This is despite the fact that Germany has experienced significant earthquakes, such as the destructive earthquakes of Düren, 18.02.1756 and Ebingen, 16.11.1911 which are reported with an intensity I_{EMS} according to the *European Macroseismic Scale*, shortly *EMS-98* [GRÜNTHAL 1998], with I_{EMS}=VIII [AMSTEIN ET AL. 2005]. In its neighbouring countries, earthquakes with even a higher intensity occurred, such as the Basel earthquake in 1356 with an intensity of I_{EMS}=IX [LANG 2002]. Within these events, historical buildings, especially churches and vaulted structures, showed their high vulnerability. Early documents underline this statement [SIEBERG AND LAIS quoted by AMSTEIN et al. 2005], [SIEBERG quoted by AMSTEIN et al. 2005]. This is one of the reasons why monumental buildings are excluded from the EMS-98 [GRÜNTHAL 1998]. As a consequence of the long return periods, churches have even been proved of having been damaged by earthquakes in areas where no seismic event was to be expected at all [KORJENKOV AND KAISER 2003].

Beside their high vulnerability, historical and monumental buildings – definitions for both given in [AUGUSTI ET AL. 2001] – exhibit large intangible values, which will be grouped within this work into cultural, social or historical values. This mixture of high vulnerability and potentially large tangible and intangible losses has to be included in the risk assessment and contributes to the challenging application of historical buildings.

If one single event was to be depicted, which comprises all the statements made above; it would surely be the partial collapse of the Basilica Superiore di San Francesco in Assisi during the Umbria-Marche earthquake in September 1997. The church is one of the most visited and venerated shrines and the collapse resulted in four dead people in the church and two completely destroyed vault paintings of the famous Italian painters of Cimabue and Giotto, which were created around the year 1300. The costs for renovation, which lasted two years during which the church was closed, were estimated to be 60 million euro [NEWS 1999]. However, these costs do not include the loss of income, as tourists were not able to enter the church. In addition to this well known example, various other examples could be addressed to raise the awareness of the problem. The following figure underlines the extreme vulnerability of historical structures to ground motions.

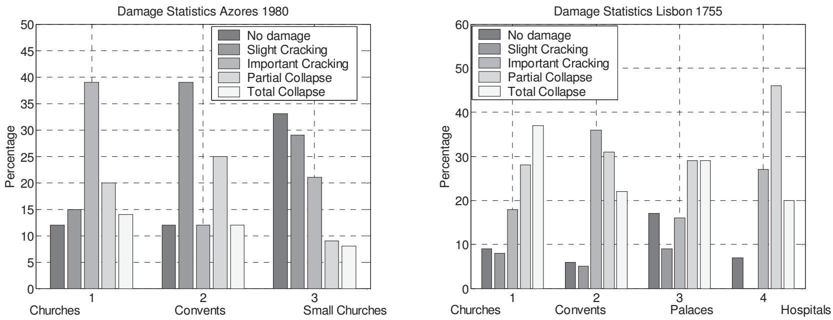


Figure 1 - 1: Damage Statistics of the 1980 Azores and the 1755 Lisbon earthquake after [OLIVEIRA 2003].

Another example is also given by [OLIVEIRA 2003] referring to the Faial/Pico earthquake that reached an intensity between VI and VIII on the Modified Mercalli Intensity [WOOD AND NEUMANN 1931] scale. During this earthquake, which occurred on July 9th, 1998 out of 27 monuments in the affected area, 59% showed heavy damage and 11% collapsed. It has to be kept in mind that this is an intensity which is also reached in regions with a comparatively low seismic hazard.

As a consequence of the aforementioned remarks it follows that the risk analysis of these structures is of great importance, not only because of the high number of structures and the public risk, but also due-to the affected cultural, social or historical values.

1.3 Defining the problem

At first the topic of this work has to be kept in mind. The term risk assessment plays a major role and it is important to differentiate clearly between the *vulnerability assessment* and the *risk assessment*. Structural engineers are usually concerned about the structural performance with respect to a given ground excitation. Performance is in the majority of all cases expressed by the pair of collapse and non-collapse. Consequently, engineers commonly speak of the risk of structural collapse, which is precisely speaking the probability of collapse. Newer concepts like *Performance Based Seismic Engineering* [PORTER 2003] introduced four different states of structural performance discretising the structural behaviour in more detail and creating the challenging task to connect numerical results to well defined damage states. Still, most projects and calculations focus solely on possible structural damage and are not taking into account the losses which might occur as a consequence of insufficient structural performance. Thus, concepts which assess only the structural reaction are usually referred to as *reliability-based concepts*. On the other hand risk – as it will be defined later – consists of the product of probability and consequences. Therefore, *risk-based concepts* include the possible impact of the structural reaction on all affected values.

Reliability-based concept: $R=P_{(f)}$ Def. 1-1

Risk-based concept: $R=P_{(Di)}\cdot P_{(C)}$ Def. 1-2

The two definitions above explain the differences between both concepts, using the following abbreviations: R is the result, $P_{(f)}$ is the probability of failure, $P_{(Di)}$ is the probability of damage of grade i and $P_{(C)}$ the probability of consequences.

As the definition explains further, it is not sufficient to perform a *scenario analysis*, which gives the result for a single input. It is also necessary to express the results in terms of probabilities to compare events with low probabilities and high consequences to those of a high probability and low consequences, which in summation might even lead to higher overall risks. The schematic description of two different kinds of probability distribution shown in figure 1-2 tries to describe the importance of including the scatter of risk and not only the mean value. While both have the same mean, risks characterized by the grey line exhibit a higher dispersion and might lead to significantly higher values.

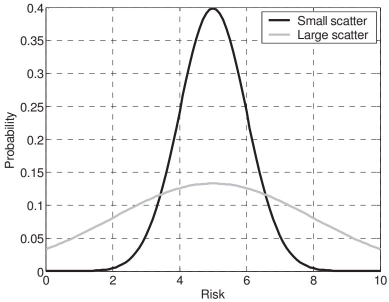


Figure 1 - 2: The importance of scatter for the assessment of the risk

Including the scatter and the probabilities of events is of special importance for the earthquake risk assessment of historical structures. A large number of uncertainties influence the outcomes. The hazard at the site is determined by the source region and source mechanisms, travel distance and medium, site effects and local characteristics, to name some of the important parameters. These uncertainties meet those which are included by the description of the material and structure. Finally, the possible losses may scatter. In the easiest way, this is explained by the number of persons, which may be present in case of structural collapse. This link between the numerical expression of structural performance, the description of the structural damage and its implications on the loss assessment is a challenging task and seldom, if at all, tackled in engineering. Nevertheless, the results would greatly affect the requirements for the structural reliability assuming that the design goal would be an identical risk and not an identical probability of failure.

Finally, it is necessary to compare risks from different kinds of fields, such as structural, sports or medical risks. Therefore, it will be necessary to express the result by such means that it is comparable to other risks in order to qualify decision makers to judge on a more objective basis and choose correct financial investments.

1.4 Scope and objective of research

Several smaller research goals contribute to the solution of the final objective of this work. This objective is to be able to express the risk inherent in historical masonry churches by means which are comparable to diverse kinds of other risks. To do so, at least some work is required in each of the substeps of the risk assessment procedure, which are shown in chronological order in figure 1-3. In more detail, this work will offer answers and contributions to the topics and related fields explained in table 1-1.

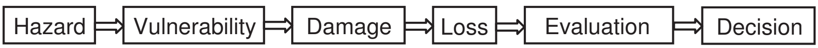


Figure 1 - 3: Flow of tasks within the risk management process

The possible impact of this work has to be regarded as very important. Not only will it enable engineers to describe possible losses of extreme events, but it will also help to determine priorities in reinforcing churches and provide advice for financial investments of regional and national authorities. Regarding pure engineering matters, results should contribute to the solution of determining important intensity parameters of earthquakes, the importance of several material parameters in comparison to each other and the description of damage from numerical results.

1.5 Overview

Work throughout the last years has revealed that risk is a new concept in civil engineering which is already used in a larger variety of disciplines. This resulted in some confusion in the application of the correct definitions of the risk management process. Hence, chapter 2 will start with an overview of the components of the risk management process and the definition of the most important tasks. Within this chapter, general information about natural and technical catastrophes will be given and a variety of existing risk parameters is going to be introduced. The following chapters will deal with components of the process shown in figure 1-3 and table 1-1. The current state of art is going to be shortly explained at the beginning of each chapter. The thesis ends at the point of the assessment of the risk in chapter 6. Because the different chapters deal only with small parts of the risk management chain, the full approach is visualized for one example in chapter 7. Chapter 8 will finally round off this work by summarizing the procedures and highlighting most important results.

HAZARD	• Determination of the probability of earthquake events • Assessing the influence of the scatter of regional dispersion of the earthquake location • Characterizing differences of natural and artificial accelerograms • Determination of the most significant earthquake intensity measures with respect to the damage in the structure
VULNERABILITY	• Assessing the influence of uncertainty in the material parameters • Determination of the sensitivity of the material parameters • Analyzing the scatter of output parameters • Enhancing the understanding of dynamic structural behaviour
DAMAGE	• Correlating analysis results with structural damage • Determination of sufficient discrete damage grades for application
LOSS	• Correlating structural damage with loss of life • Correlating structural damage with Cultural, Social and Historical (CSH) Losses
ASSESSMENT	• Definition of risk classes • Offering a first approach to relate CSH values to loss of human life • Comparison of risks to other types of buildings • Comparison to other types of risks

Table 1 - 1: Research tasks to be performed

Chapter 2

The concept of risk management in civil engineering

2.1 The process of risk management

2.1.1 Introduction

Risk management is a process gaining more importance and increasing attention in civil engineering in the last years. Finding the roots is a difficult task. One might argue that it was already included, albeit in a rather unique form, within the code of Hammurabi [HAMMURABI 2007]:

> "If a builder has built a house for someone, and does not construct it properly, and the house which he has built falls in and kills the owner, then that builder shall be slain." Def. 2-1

In linguistics the term *risk* may be related to the Italian *rischiare*, meaning *daring* or to the Latin word *risicare* which means *to steer around a cliff* [PLAPP 2003] [PROSKE 2004]. In German language the term is first referred to in sources throughout the 16th century by merchants discussing possible investments [PLAPP 2003]. In modern applications the first ideas of risk management may be found in early economic theories around the 1920´s [KNIGHT 1921]. These are related to insurance and stock market decision theory. Roughly within the 1950´s the concept was adopted by health sciences [NAC 1960]. At first it was mainly used to describe the possible impacts of nuclear radiation on human life, but it was extended soon to find rules for food and health regulations. Within the evaluation of health risks, the term *de minimis risk* [WHIPPLE 1987] was introduced. This term, which is explained further in chapter 2.3.3, resulted from the first legal attempts to agree on an acceptable level of risk. Also, health scientists offered the first definition of risk management:

> "A decision-making process involving the consideration of information of political, social, economic and technological nature, in addition to data concerning risks, in order to develop, analyse and compare regulatory options; the goal of this process is to select the most appropriate response with respect to the potential risks that may pose a chronic threat to health [NRC 1983]." Def. 2-2

Determining an exact date, when the ideas of risk management have found their way into the various disciplines of civil engineering for the first time is hardly possible due-to the large variety of tasks within the civil engineering itself, but also because the diverse tasks included in the definition of risk management. What may be noticed though is that around the late 1990´s the focus on possible losses and acceptable risk criteria on the basis of risk-based and not only reliability based approaches has received significantly more attention and several research groups have been raised. Due-to the large variety of topics to which the task of risk management was applied, some confusion resulted, because several definitions for similar principles exist. The definition of risk serves well as an example. While in colloquial use the word *risk* is sometimes applied for the hazard itself, as in ”within this area there is a certain risk of an earthquake to occur”, other definitions are frequently found within recent publications:

Risk = *probability* times *damage* [GROTHMANN AND REUSSWIG 2006] Def. 2-3
Risk = *probability* times *consequences* [SCHNEIDER 2002] Def. 2-4
Risk = *hazard* times *vulnerability* times *exposure* [KRON 2002] Def. 2-5

All have in common the combination of the probability or frequency of an event and its implication on the considered system. Now, although this adaptability of the definition is certainly a key strength, it creates confusion. Regarding the outcome of risk based calculations the units describing the risk have to be the same, no matter what definition is utilized. Thus, it is crucial not to concentrate on the definitions themselves at first, but on the process to realize the connection of the different parts. In this way it will not only be possible to clarify misunderstandings related to different definitions, but furthermore an integrated approach will be achieved, which is applicable in every discipline. After this is done, the definitions for major parts of the overall risk management process will be derived and used further in this study.

2.1.2 Risk management framework

At the beginning the components of the risk management process have to be identified. A very general definition was already offered in the chapter 2.1.1. Three additional definitions are offered here:

“The systematic application of management principles, procedures and practices to the task of analysing, assessing and controlling of risks [DIN 14971].” Def. 2-6

“Systematic application of policies, procedures and practices to the tasks of identifying, analysing, evaluating, treating, and monitoring risk [AS/NZS 4360].” Def. 2-7

“The systematic application of quality management policies, procedures, and practices to the tasks of assessing, controlling, communicating and reviewing risk [WEB 1].” Def. 2-8

Despite the similarities which exist for these definitions it is nonetheless obvious that they are applied to different fields and the language applied differs in details. As a summary, the tasks of identifying, analysing, assessing, evaluating, controlling, treating, monitoring, communicating and reviewing were addressed. Now, stating that the assessment may be subdivided into the analysis and the evaluation and that controlling and monitoring are equal to a constant review of risk, the following definition for risk management is derived:

"Risk management is defined as the systematic application of management policies, procedures and practices to the tasks of identifying, assessing, treating, communicating and reviewing risk." Def. 2-9

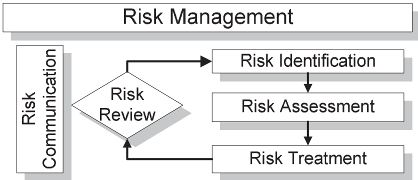


Figure 2 - 1: The process of risk management and its first order subdivisions

Displayed in a picture, the definition above would look like shown in figure 2-1. The three main components are the *risk identification*, the *risk assessment* and the *risk treatment* which are performed chronologically throughout the *risk management* process. Within the *risk review* all new information, knowledge and experience about the risk is included and the evolution of the risk constantly monitored. Accompanying all the steps is the risk communication, which is an expression for the flow and exchange of information of all persons actively or passively participating, or more precise being affected in the risk management process. These five elements give only a rough subdivision of the risk management process. For engineers, the most important tasks lie within the parts of risk assessment and risk treatment. Thus these items will be subdivided and explained further within the following text.

2.1.3 Risk assessment

Risk assessment consists of two general parts: *risk analysis*, in which the risk is calculated, and the *risk evaluation*. The risk evaluation compares the results of the risk analysis to the possible outcomes for other events or structures. This comparison results in the creation of risk classes and in grading of the structures according to the overall threat. In this part, risk classes or acceptable risk levels may be defined. The risk analysis can be split up further into the three components of *hazard analysis*, *damage determination* and the *loss assessment*. The new components are shown in the following figure.

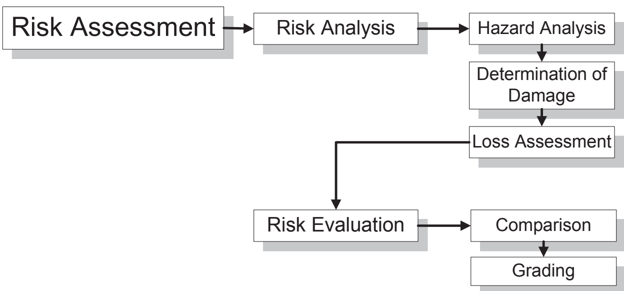


Figure 2 - 2: The components of the risk assessment

Hazard analysis consists of the three parts of *hazard identification*, *determination of the relevant intensity levels* and the *time dependent probabilities of occurrence*. It is important to realize that the intensity levels are affected by a number of factors, e.g. the location of occurrence and the path or medium through which the effects of the hazard proceed. This is well expressed by the *source-pathway-receptor* model, which is briefly explained in chapter 3.2.1. Next step in the risk analysis procedure is the *damage determination*. In risk assessment it is necessary to distinguish between the damage, which is related directly to the structure or the system analysed, and the loss, which occurs as a consequence of the structural damage. Structural damage captures the material harm and may be expressed by a larger variety of measures, e.g. degree of moisture, crack width, spectral displacement or percentage of structural collapse. It is not expressed in monetary terms. The damage depends on the structural properties and the intensity of a hazard for a given site. The relation between the hazard intensity and the resulting damage is called the *structural vulnerability*. The structural vulnerability is a specific characteristic of an element that indicates the susceptibility of a structure towards the impact of a hazard. Structural vulnerability is to tell apart from *system vulnerability*, which directly relates the hazard to the *loss*. The loss is the accumulation of all *direct and indirect consequences* which might result from a structural damage. Direct consequences occur simultaneously to the time the disaster takes place or by immediate follow-on destruction. Indirect consequences occur with a time shift as a result of the direct consequences. They may be interpreted as follow-up costs that result from the element at risk not being able to carry out its designated functionality within the system after the disaster has occurred. Moreover, as shown in figure 2-3, direct as well as indirect consequences are to be further subdivided into the different types of losses of which the most important are *economic*, *human*, *CSH*, i.e. cultural, social and historical and *ecological*. Because it is possible to assign a monetary value only to economic consequences in a direct way, they will also be referred to as *tangible*. All other classes of consequences are termed *intangible*. A schematic overview is included in the following diagram.

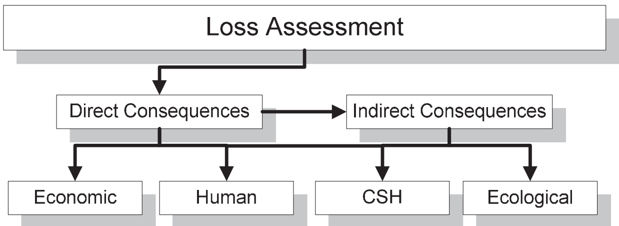


Figure 2 - 3: Schematic description of the loss assessment

The differentiation of damage and loss explains already the differences between definition 2-3 and 2-4. To clarify this, in the following the terms of *structural risk* and *total risk* will be used according to definitions 2-10 and 2-11.

Structural risk = *probability* times *structural damage*	Def. 2-10
Total risk = *probability* times *loss (i.e. consequences)*	Def. 2-11

If the risk assessment is performed for a system of various elements, such as several buildings, we speak of a *system* if all elements are considered. The single element is usually referred to as the *Element at Risk*, or simply *EaR*. An EaR may be seen as a single or a group of persons or objects within the predefined system that are susceptible and exposed to the impact of a hazard. The *exposure* is another term frequently used in risk management. It is commonly used by the United Nations to assess the impact of disasters on larger systems [UN 2004]. To be exposed is the act of being subjected to an influencing experience. The exposure includes the inventory of the Element at Risk and the local effects of the hazard. It is object and hazard related. In figure 2-4 the exposure is assigned to the hazard, but it could also be connected to the loss. Because of the diverse definitions, this work tries to avoid the term exposure.

2.1.4 Risk treatment

The risk assessment results in the decision about what action is required, in other words, how the risk is handled or treated. There are four major types of reaction to the outcome of the risk assessment. At first the risk might be considerably too high. In this case the risk is *rejected*, i.e. the project has to be abandoned or at least a different site or time for the execution must be evaluated. Next, the risk might fall within expected thresholds, so that no action needs to be taken into consideration, the risk is *accepted*. If the risk happens to fall within these two extremes the risk might either be *transferred*, e.g. via insurances, or *mitigated*. A combination of actions is also possible.

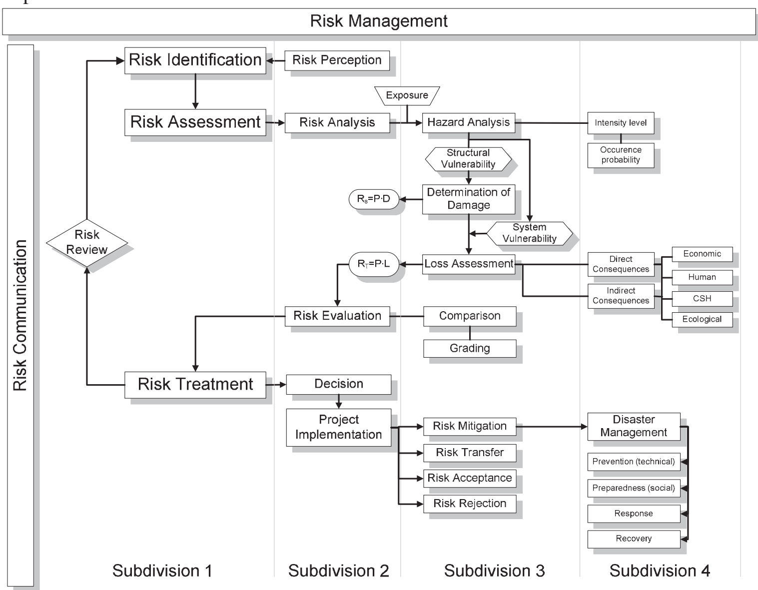


Figure 2 - 4: The complete risk management process

If the risk is mitigated it can in turn be distinguished between four aspects that are also defined as *disaster management* [UNESCO 1999]: *prevention*, *preparedness*, *response* and *recovery*. Of major importance are the pre-disaster interventions of prevention, which comprises all technical measures taken in order to reduce a risk and preparedness, which includes all social activities, e.g. disaster plans. Response covers all activities taken immediately after the disaster up to the organization and communication of all participating agencies. Recovery determines the time from the immediate reaction until the point the pre-disaster status quo is reached again. The complete process of the risk management including all details discussed is shown on the previous page. A summary of all the definitions used may be found in appendix A. They are based on an internal report of [PLIEFKE, SPERBECK, URBAN 2006].

2.1.5 Calculation of risk

In chapter 2.1.1 the different existing definitions for the calculation of the risk were shortly described. Given the background of chapters 2.1.2 to 2.1.4 the different approaches may now be compared and units assigned to each of them. The difference between structural risk and total risk was already explained. The resulting units would be:

Structural risk = [Damage measure/time unit]
Total risk = [Loss measure/time unit]

Whereas the time unit is commonly taken as a year, the damage measure and loss measure may differ: loss of life, loss of money, degree of humidity or excess of a certain temperature. If the risk is defined as in definition 2-5 the following units are used:

Hazard = [Intensity measure/time unit]
Vulnerability = [Damage measure/intensity measure] or
[Loss measure/intensity measure]
Exposure = [Dimensionless factor between 0 and 1]

One might argue which concept is the best and most suitable for the given problem. For reasons of simplicity and due-to difficulties in handling the exposure, definitions 2-3 and 2-4 are clearly favoured within this work.

2.2 Background data on catastrophe occurrence

2.2.1 Natural catastrophes

It is important to explain the diverse kinds of catastrophes relating to low probability - high consequence events, which hereafter will be called *LPHC* events. To understand the way risks of extreme events are perceived and described, at first a short overview about the different types of catastrophes due to civil, anthropogenic or natural impact will be given. The last years have exhibited an increase of natural catastrophes, which is described in table 2-1, created after data presented in [MUNICHRE 2006]. The last column expresses the increase in terms of a factor, if the 1960´s are compared to the last 10 years of monitoring and reviewing natural catastrophes. The increase is quite clearly visible.

Decade	**1950-1959**	**1960-1969**	**1970-1979**	**1980-1989**	**1990-1999**	**Last 10 years**	**Last 10:60s**
Number of events	21	27	47	63	91	57	2.1
Overall losses[1]	48.1	87.5	151.7	247.0	728.8	575.2	6.6
Insured losses[1]	1.6	7.1	14.6	29.9	137.7	176.0	24.8

[1] Losses in US$ bn (2005 values)

Table 2- 1: Comparison of losses of decades 1950 until 2005 [MUNICHRE 2006]

The *Emergency Disasters Data Base* [EM-DAT 2006] provides similar results, as pictured in figure 2-5. To be included into this database, the catastrophe has to cause one of the following four criteria: 10 or more people reported killed, 100 or more people reported affected, a call for international assistance or the declaration of a state of emergency.

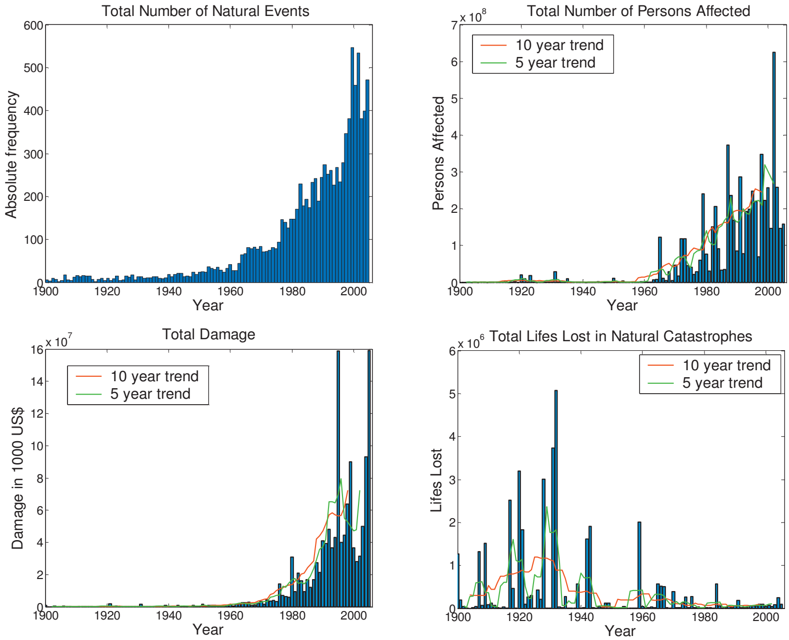


Figure 2 - 5: Number and effects of natural catastrophes 1900-2005

The increasing number of events is clearly visible as is the number of persons that are affected or the damage resulting from the catastrophes. Since it is still disputed whether the last five to ten years have still shown an increase, the five and ten year trends are also shown in the picture. Concerning the number of lives lost it can be seen that extreme numbers were reached during the earlier years of this century; this is because droughts and infections are included in this data. Still, the decrease of loss of life can to some degree also be ascribed to safety regulations and improved safety codes. The increasing impact of disasters is not a consequence of weaker buildings, but more a result on the rising number of large and crowded cities located in highly exposed regions. In this manner comparatively small events may lead to significant immediate losses. If important infrastructural facilities are affected losses may even increase further due-to indirect effects. This is essential to keep in mind, since it clearly explains the difference between a natural event and a catastrophe, which might simply be seen as a natural event happening in a crowded location. Owing to this explanation, the number of events itself must not necessarily be increasing. Instead, it is the number of high exposed cities. The impact of disasters on insured losses and loss of life is described by data according from [SWISSRE 2006] in tables 2-2 and 2-4.

Insured Loss[1]	Victims[2]	Date (Start)	Event	Country
45000	1326	24.08.2005	Hurricane Katrina	US, Gulf of Mexico
22274	43	23.08.1992	Hurricane Andrew	US, Bahamas
18450	2982	11.09.2001	Terror Attack	US
11684	61	17.01.1994	Northridge Earthquake	US
10000	124	02.09.2004	Hurricane Ivan	US, Caribbean Sea
10000	34	20.09.2005	Hurricane Rita	US, Gulf of Mexico
8272	35	16.10.2005	Hurricane Wilma	US, Mexico, Jamaica
8097	24	11.08.2004	Hurricane Charly	US, Cuba, Jamaica
6864	51	27.09.1991	Typhoon Mireille	Japan
6802	95	25.01.1990	Winter storm Daria	France, UK, NL et al.

[1] In million 2005 USD, incl.: property and business interruption, e.g.: liability and life insurance losses. [2] Dead and missing

Table 2- 2: The 10 most costly insured losses 1970-2005

Victims[1]	Insured Loss[2]	Date (Start)	Event	Country
300000	-	14.11.1970	Storm and Flood	Bangladesh
255000	-	28.07.1976	Earthquake (M 7.5)	China
220000	2068	26.12.2004	Earthquake (M_w 9), Tsunami	Indonesia, Thailand et al.
138000	3	29.04.1991	Tropical Cyclone	Bangladesh
73300	-	08.10.2005	Earthquake (M 7.6)	Pakistan, India et al.
66000	-	31.05.1970	Earthquake (M 7.7)	Peru
50000	172	21.06.1990	Earthquake (M 7.7)	Iran
26271	-	26.12.2003	Earthquake (M 6.5)	Iran
25000	-	16.09.1978	Earthquake (M 7.7)	Iran
25000	-	07.12.1988	Earthquake (M 6.9)	Armenia

[1] In million USD, indexed to 2005, including: property and business interruption, excluding: liability and life insurance losses. [2] Dead and missing

Table 2- 3: The 10 worst catastrophes in terms of victims 1970-2005

It can be seen that storms are dominating the insured losses and largely affecting the United States of America, whereas with respect to loss of life, earthquakes in the Far and Middle East are the most important events. A more detailed overview would show how regional differences are affecting the type of catastrophe. Since this would exceed the scope of this study the interested reader is referred to the disaster database [EM-DAT 2006].

Disaster	Date	Killed		Disaster	Date	Affected		Disaster	Date	Damage[1]
Earthquake	28.12.1908	75 000		Flood	07.10.1970	1 301 650		Earthquake	23.11.1980	20 000 000
Earthquake	13.01.1915	29 980		Flood	03.11.1966	1 300 000		Earthquake	26.09.1997	4 524 900
Temperature	16.07.2003	20 000		Earthquake	23.11.1980	400 000		Earthquake	06.05.1976	3 600 000
Earthquake	23.11.1980	4 689		Earthquake	06.05.1976	218 222		Flood	03.11.1966	2 000 000
Earthquake	08.09.1905	2 500		Flood	14.11.1951	170 000		Flood	19.06.1996	1 000 000
Slides	09.10.1963	1 917		Earthquake	28.12.1908	150 000		Flood	29.08.2003	914 490
Earthquake	23.07.1930	1 883		Earthquake	15.01.1968	55 563		Earthquake	31.10.2002	796 000

[1] In 1000 US$

Table 2- 4: Top seven natural events in Italy 1900-2005 for lifes lost, affected persons and damage

Regional and national differences of disasters and their impact shall only be explained on the base of the top seven disasters for Italy and Germany since both lie in the focus of this study. Data are again taken from [EM-DAT 2006] and shown in tables 2-4 and 2-5.

Disaster	Date	Killed		Disaster	Date	Affected		Disaster	Date	Damage[1]
Temperature	Aug. 2003	5250		Flood	11.08.2002	330 000		Flood	11.08.2002	11 700 000
Wind Storm	Feb. 1962	347		Flood	21.12.1993	100 000		Wind Storm	Jan. 1990	4 535 3000
Wind Storm	02.01.1976	82		Flood	22.05.1999	100 000		Temperature	Aug. 2003	1 650 000
Wind Storm	Jan. 1990	64		Flood	10.01.1995	30 000		Wind Storm	26.12.1999	1 600 000
Wind Storm	12.11.1972	54		Flood	04.07.1997	15 000		Wind Storm	02.01.1976	1 300 000
Temperature	04.01.1997	30		Flood	26.03.1988	3 500		Flood	10.01.1995	1 000 000
Flood	11.08.2002	27		Earthquake	13.04.1992	1 500		Wind Storm	12.07.1984	1 000 000

[1] in 1000 US$

Table 2- 5: Top seven natural events in Germany 1900-2005 for lifes lost, affected persons and damage

2.2.2 *Technical catastrophes*

Since the early 1960's another large group of catastrophes, the technical catastrophes, receives wide attention. The catastrophes caused by technical events may be roughly divided into industrial catastrophes and transportation catastrophes. Within the industrial ones, the event most closely related to this study is the collapse of a structure of which the largest influence can be assigned to dam breaks. The subsequent tables give reference about collapses of dams and buildings. Further data may be found in [EM-DAT 2006] or [PROSKE 2002].

Year	Country	Location	Lifes Lost	Affected Persons
1979	India	Morvi	1335	150000
1923	France	Gleno	600	0
1959	France	Malpaset	412	6000
1928	United States	Saint Francis	400	0
1993	China P Rep	Conghe county	370	33136
1985	Italy	Cavalese-Stava	329	30
1972	United States	Canyon Lake	240	0
1976	United States	Canyon Lake,	237	0
1959	Spain	Vega de Tera	135	500
1972	United States	Lake Barcroft	125	0

Table 2- 6: Major disaster due-to dam break [EM-DAT 2006], [PROSKE 2002]

Year	Structure	Country	Location	Lifes Lost	Affected Persons
1978	Temple	Guyana	Jonestown	900	0
1995	Department store	Korea Rep	Seoul	458	922
1982	Luzhniki Stadium	Soviet Union	Moscow	340	0
1980	Building	Colombia	Sincelejo	165	500
2005	Garments factory	Bangladesh	Palash Bari (Near Dacca)	151	100
1993	Hotel 'Royal Plaza'	Thailand	Nakhov Ratchasima	135	270
1984	Bridge	India	Kerala	125	0
1990	School	Nigeria	Port Harcourt	100	0
1989	Football stadium	United Kingdom	Sheffield	95	200
2004	Building	Turkey	Konya	94	28

Table 2- 7: List of collapses of buildings [EM-DAT 2006]

Considering the trends in industrial catastrophes, similar conclusions as for natural catastrophes may be drawn. Numbers were increasing steadily between 1970 and 1990. Since 1990 a constant number is observed, which is based on decreasing numbers in the industrial countries and increasing ones in developing countries. The bulk is carried by explosions and fires. Nuclear catastrophes are also included in these numbers. During the time this chapter is written the 20th anniversary of the maximum credible event in Tschernobyl takes place and it is interesting to see that it does not show significant influence if viewed with respect to all other industrial events happen worldwide. Even so, this event is remaining a politically controversially discussed matter and the only conclusion in this thesis shall be that whereas in general technical catastrophes are more locally focused, affecting less people and thus covering smaller regions, this was an event having effects on a global scale equal to those of the natural catastrophes. Also, people are involuntary subjected to this type of catastrophe. This is a factor largely influencing the risk perception as explained in chapter 2.3.1.

Within the technical catastrophes, traffic and transportation contribute as a second factor to the amount of lifes lost and damage. Reviewing the safety of transportation, discussions usually culminate in the question which transportation system is the safest. This question arises because the risk measures are based on different assumptions. Naturally, airplanes are safest if the number of deaths per travelled kilometre is taken. In that case, the probability of dying is $4.19 \cdot 10^{-9}$ for planes while the risk of an accident – not necessarily including deaths – is $4.67 \cdot 10^{-6}$ for cars and $2.13 \cdot 10^{-5}$ for trains [PROSKE 2002]. Neglecting that these numbers cannot be compared easily because they relate accidents to deaths, airplanes are the least safe transportation method, if the comparison is based the number of travels instead of the number of travelled kilometres. It is neither the purpose nor the intention of this study to solve this question. In its place, the example shall serve as a motivation and justification for the creation of the risk measures explained in remaining part of this chapter as well as for their advantages and drawbacks.

2.2.3 Other causes of death

Several other causes of death may be named. The three most important types are health conditions, social surroundings, which include poverty and violence in all shapes, i.e. also wars, and sports activities. In general, these deaths happen at a larger frequency, but because they are neither focused in time nor in space they are usually perceived as less important.

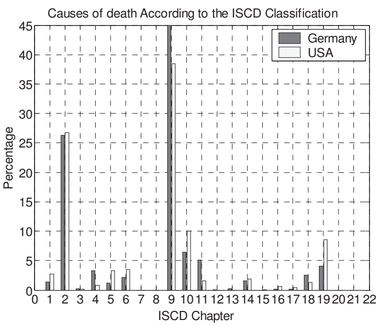
Although several examples were given for the impact of natural and technical catastrophes, defects in the functional ability of our body are in most cases the cause of death. Those deficiencies could be caused by diseases or pandemics, but typically they are the result of advanced age. To obtain information about the more frequent causes of death, information is usually provided in a very detailed manner by the National Statistical Offices. They classify the causes after the *International Classification of Diseases and Related Health Problems*, shortly called ISCD categories. The results from Germany for the year 2004 and the United States in year 2003 are shown in table 2-8 after data from [DESTATIS 2005] and [US HEALTH 2005]. As expected, health related problems – especially cancer – and defects of the circulatory system dominate the results.

A closer look at the data reveals also the importance of social risks. Nearly fifty percent of the deaths falling into chapter V, table 2-8, in Germany are due to excessive consume of alcoholics. In contrast, 2003 data for the US reveal that 700 persons died as a consequence of the unintended discharge of firearms, cf. table 2-9. This does not include the homicides and suicides.

ISCD Chapter	Source	Blocks	Germany		USA	
			Total Number	Percentage	Total Number	Percentage
I	Infectious/Parasites	A00-B99	11062	1.35	64661	2.74
II	Neoplasm	C00-D48	214863	26.26	633104	26.8
III	Blood diseases	D50-D89	2054	0.25	4594	0.19
IV	Metabolic diseases	E00-E90	27041	3.3	18157	0.77
V	Mental disorder	F00-F99	9516	1.16	77557	3.28
VI	Nervous system	G00-G99	17675	2.16	82184	3.48
VII	Eye disease	H00-H59	3	0	0	0
VIII	Ear disease	H60-H95	11	0	0	0
IX	Circulary system	I00-I99	368472	45.03	907180	38.4
X	Respiratory system	J00-J99	52500	6.42	235935	9.99
XI	Digestive system	K00-K93	42213	5.16	36416	1.54
XII	Skin	L00-L99	587	0.07	0	0
XIII	Musculoskeletal system	M00-M99	1981	0.24	0	0
XIV	Genitourinary system	N00-N99	13246	1.62	44093	1.87
XV	Pregnancy and child birth	O00-O99	37	0	545	0.02
XVI	Perinatal period	P00-P96	1443	0.18	14378	0.61
XVII	Congenital malformations	Q00-Q99	1576	0.19	10518	0.45
XVIII	Not classified	R00-R99	20682	2.53	31444	1.33
XIX	Injury and poisoning	S00-T98	33309	4.07	201676	8.54
XX	External causes*	V01-Y98	0	0	0	0
XXI	Factors influencing health	Z00-Z99	0	0	0	0
XXII	Special purposes	U00-U99	0	0	0	0
	Total		818271	100	2362442	100

* External causes might repeat Chapter XIX, compare table 2-9

Table 2 – 8: Causes of death in Germany 2004 and the USA 2003 according to the International Statistical Classification of Diseases and Health Related Problems.



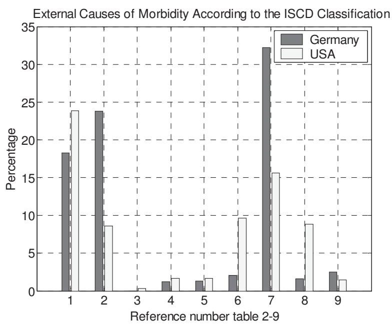


Figure 2 - 6: Causes of death in percentage with reference to table 2-8 (left diagram) and table 2-9 (right diagram)

A closer look at the external causes of death is given as an excerpt in figure 2-6 and table 2-9. The presented data intends to give background information about what types of risks and influences are major causes of death and at which age they mostly occur. It is especially important, if the acceptable risk bases for societal risk are to be established, which will be explained in the following chapter.

No.	Source	Germany		USA	
		Total Number	Percentage	Total Number	Percentage
	All other diseases (residual)	33309		201676	
1	Transport accidents (V01-V99,Y85)	6087	18.27	48071	23.84
2	Falls (W00-W19)	7913	23.76	17299	8.58
3	Accidental discharge of firearms (W32-W34)	0	0	730	0.36
4	Accidental drowning and submersion (W65-W74)	401	1.2	3306	1.64
5	Accidental exposure to smoke, fire and flames (X00-X09)	446	1.34	3369	1.67
6	Accidental poisoning and exposure to noxious substances (X40-X49)	680	2.04	19457	9.65
7	Intentional self-harm (suicide) (*U03,X60-X84,Y87.0)	10733	32.22	31484	15.61
8	Assault (homicide) (*U01-*U02,X85-Y09,Y87.1)	526	1.58	17732	8.79
9	Complications of medical and surgical care (Y40-Y84,Y88)	831	2.49	2855	1.42

Table 2 – 9: Causes of death in Germany 2004 and the USA 2003 according to the International Statistical Classification of Diseases and Health Related Problems only regarding chapter XX (excerpt).

2.3 Measuring and comparing risks

2.3.1 General remarks

As already described, risk is defined as the product of the probability of an event and its consequences. The outcome is often measured by loss of life or money per year. Although this suggests that risk is rather easy to calculate, the opposite is true. Whereas scientific calculations claim to reflect the reality objectively, individuals often have difficulties in understanding the outcomes. One reason is that most people have problems in handling terms of probability [PLAPP 2003]. Thus, to be able to communicate risks, results are often expressed in deterministic ways, e.g. worst case scenarios, helping to find different levels of acceptance. This, on the other hand, greatly restrains the possibilities of risk assessment. In order to overcome these restrictions, a number of risk parameters were introduced into the scientific society with the purpose to relate and communicate risks.

Before explaining the most common parameters, additional remarks need to be given. It is a fact that risks are often perceived differently by individuals and experts. It might be said that one‘s perception equals one’s reality, resulting in diverse attitudes towards risk. This is why risks due-to smoking or sports are accepted easier than for example being involuntary exposed to the risk of an adjacent nuclear power station. This example does not only explain the importance of individual risk perception, but also the difference between what is called individual risk, which is freely chosen and societal risk, for which the public has to determine acceptable levels. Both subjects shall be briefly clarified.

With respect to the individual risk perception [SKJONG AND WENTWORTH 2007] present 8 major influences with reference to [LITAI 1980] on the individual risk perception, the so called risk conversion factors.

Characteristics	**Scale**	**RCF**
Volition	Voluntary-Involuntary	100
Severity	Ordinary-Catastrophic	30
Origin	Natural-Man made	20
Effect Manifestation	Delayed-Immediate	30
Exposure Pattern	Regular-Occasional	1
Controllability	Controllable- Uncontrollable	5-10
Familiarity	Old-New	10
Necessity	Necessity-Luxurious	1

Table 2- 10: Risk conversion factors according to [SKJONG 2002] and [LITAI 1980]

The meaning of the risk conversion factor is to represent the bias of two risks, which are ranked identical in terms of real losses, but only differ in the scale mentioned in the middle column. For example, man made catastrophes that result in the same amount of losses are judged by a factor of 20 higher than a risk occurring due-to a natural event. Some of these points are also addressed by [PLAPP 2003], who also summarizes some of the main factors influencing the individual risk perception. These factors are listed in the following table.

Heuristics	**Description of the effect**
Availability	Events being remembered closely are assumed to be more probable than events not being cognitively available.
Anchoring effect	Probabilities for events are adjusted to the perceived importance of the information given.
Effect of representativeness	Out of small samples, i.e. a short period of experience of events, the basic population is concluded with a large error size. Personal experience is rated higher than information based on frequency.
Avoidance of cognitive dissonance	Information that is questioning guessed probabilities, which are part of an existing “system of believes” is ignored.
Gamblers fallacy	In judging frequencies, regularities are constructed, which in reality do not exist. Example Card Game: “After three times spades, diamonds has to follow next”.
Effect of familiarisation	The more continuous and evenly distributed the losses are, the easier are catastrophic occurrences are excluded and the size of mean losses underestimated.

Table 2- 11: Heuristics and their effect on individual risk perception

Other effects also play a role of which the most important are the social-cultural background, and the effect of optimistic assessment, which is for example explained in [PROSKE 2002]. Summarizing these comments, interesting results become evident. Individuals turn to set themselves different preferences than those the public has to offer. This leads to some of the problems already mentioned, such as the concentration of population in areas with high natural risks, which is depicted in figure 2-7 after [PROSKE 2004]. It remains a societal task to provide sufficient safety, for example by regulation of building codes.

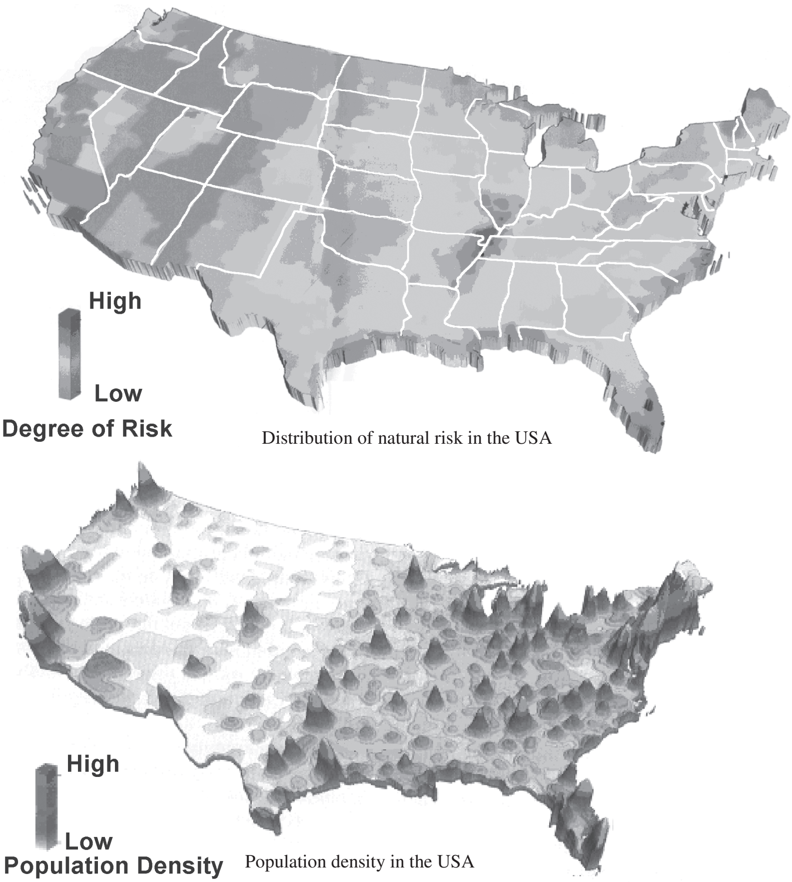


Figure 2 - 7: Population density vs. distribution of natural risks in the USA, modified after [PROSKE 2004]

Building codes, not necessarily only those dealing with the construction of buildings, set target reliabilities of structures, which are based on the maximum allowable probability of collapse of a structure. This will be the first risk measure to be introduced in the subsequent text.

2.3.2 *Probability of collapse and target reliabilities*

Most building codes include a very basic risk-based approach. For example in the German GruSiBau [GRUSIBAU 1981] three safety classes are introduced for which the collapse probabilities are given as follows.

Class	Description	Operative collapse probability per year
1	No danger for persons and low economic consequences	$1.34 \cdot 10^{-5}$
2	Danger for persons and / or considerable economic consequences	$1.30 \cdot 10^{-6}$
3	High importance of the structure for the public	$1.00 \cdot 10^{-7}$

Table 2 – 12: Operative collapse probabilities per year after [GRUSIBAU 1981]

Since most of the structures should fall within the category 2 and precise information about the type of building or considerable economic losses is not given, this might still be graded into a reliability-based concept. An improvement, especially for historical structures, was developed by [SCHUEREMANS 2001]. Three different formulas for the assessment of the allowable probability of failure are given.

$$p_{fT} = 10^{-4} \cdot S_c \cdot \frac{t_L}{n_p} \tag{2.1}$$

Where p_{fT} is the probability of failure, t_L the design service life, S_c a social criterion and n_p the average number of persons in the immediate neighbourhood of the building or in the building itself. This formula is based on data given in [CIRIA 1977]. The values for all factors are given in table 2-13. A second formula is given with reference to [ALLEN 1991].

$$p_{fT} = 10^{-5} \cdot \frac{A_c}{W} \cdot \frac{t_L}{\sqrt{n_p}} \tag{2.2}$$

In this formula, A_c represents an activity factor and W the type of collapse. Again, all values for the factors are cited in table 2-13. The last is a value specifically derived for the assessment of the structural safety of historical buildings.

$$p_{fT} = 10^{-4} \cdot S_c \cdot \frac{t_L}{n_p} \cdot \frac{A_c}{W} \cdot C_f \tag{2.3}$$

The newly introduced factor is C_f to consider also the economic effect. All the given values are shown in the following table. Although detailed information is still missing, this approach offers more possibilities to adjust the structural safety to the risk. No information was found about how the single values for the factors were derived.

Social criterion factor Sc	Places of public assembly, dams, historical buildings of great importance for mankind, e.g. listed by UNESCO	0.005
	Domestic buildings, offices, trade buildings, industrial buildings, listed historical buildings	0.05
	Bridges	0.5
	Towers, masts, off-shore structures	5
Activity factor Ac	Post disaster activity	0.3
	Normal activity building	1.0
	Normal activity bridge	3.0
	High exposure structures (e.g. offshore)	10.0
Warning factor W	Fail-Safe condition	0.01
	Gradual failure with some warning likely	0.1
	Gradual failure hidden from view	0.3
	Sudden failure without previous warning	1.0
Economic factor Cf	Not serious	10
	Serious	1
	Very serious	0.1

Table 2 – 13: Factors to be used in formulas 2.1 – 2.3

Various other numbers for maximum collapse probabilities may be found in literature. Only a small number of them are summarized in the following table.

Project	Failure probability	Source
Hermes Space Travel Project	10^{-4}	[ALTAVILLA ET AL. 2000]
Nuclear failure per year and power station in Germany	$3.8 \cdot 10^{-7}$	[GRS 1999]
Mean number of persons < 0.1 and low economic consequences	10^{-3}	[CEB 1976]
Mean number of persons >10 and high economic consequences	10^{-7}	[CEB 1976]

Table 2 – 14: Failure probabilities

Best known is the value of 10^{-6}, although target values may differ up to an order of magnitude below and up to three orders of magnitude above, indicating that the 10^{-6} is seldom met in application. Despite the usefulness of these numbers, no information is included about the possible loss of life and its probability. This might be seen if mortality rates are defined.

2.3.3 Mortality rate or the probability of death

The simplest measure to describe the risk of human death is the *mortality rate*. Or, since this term is used in medical science with a different meaning, it is better to speak more generally of the *probability of dying* as the consequence of an event. This probability is normally expressed by dividing the number of deaths per year through the size of the sample population – normally a country. Using the data presented in table 2-9 and taking Germanys population size of 82 508 000 persons in the last quarter of 2004 from [DESTATIS 2005], the mortality rate due-to transportation accidents in Germany is 6087/82 508 000. This equals $7.3 \cdot 10^{-5}$. What is often done, but should be avoided, is to compare probabilities of dying and target reliabilities of structures. A good overview of this and other sources may be found in [PROSKE 2002]. Some data are presented in table 2-15.

Action or Reason leading to death	Relative mortality rate per year
Death of borne children in Mali	$1.2 \cdot 10^{-1}$
German soldier in World War II	$7.0 \cdot 10^{-2}$
General mortality rate in the USA	$9.0 \cdot 10^{-3}$
Flying (Crew)	$1.2 \cdot 10^{-3}$
Car accident (USA)	$2.4 \cdot 10^{-4}$
Construction worker (2200 hours/year)	$1.5 \cdot 10^{-4}$
Household	$1.0 \cdot 10^{-4}$
Civil engineer	$1.9 \cdot 10^{-5}$
Natural catastrophe in the USA	$1.4 \cdot 10^{-6}$
Lightning (U.K.)	$1.0 \cdot 10^{-7}$
Earthquake (USA)	$5.1 \cdot 10^{-8}$
Extinction event in the history of the earth	$1.1 \cdot 10^{-8}$
Impact of meteorite	$6.0 \cdot 10^{-11}$

Table 2- 15: Probability of dying

The mortality rate is also used to derive target values for individual risks applied by society, especially for working and construction facilities. Acceptable risk values are also referred to as *de minimis risk*, which comes from the Latin *de minimis non curat lex*, which could be translated as *the law does not concern itself with trifles*. This value is often taken as $1.0 \cdot 10^{-6}$ although the roots cannot be determined any more. A very interesting research about the origin of this value was performed by [KELLY 1991] carrying the title "The myth of 10^{-6} as a definition of acceptable risk". The title is already indicating the result of the study. Nowadays, more refined definitions are available as the examples in table 2-16 show.

Acceptable risk of one person dying within one year	Relative mortality rate per year	Source
Acceptable risk in British heavy industry (old value)	$4 \cdot 10^{-3}$	[PATÉ-CORNELL 1994]
Acceptable risk in British heavy industry (new value)	$2 \cdot 10^{-3}$	[PATÉ-CORNELL 1994]
Acceptable risk on British oil platforms	$1 \cdot 10^{-3}$	[PATÉ-CORNELL 1994]
Acceptable risk for old buildings	$1 \cdot 10^{-4}$	[PATÉ-CORNELL 1994]
Maximum tolerable risk for public	$1 \cdot 10^{-4}$	[HSE 2001]
Acceptable risk	$1{,}1 \cdot 10^{-5}$	[COMAR 1997]
Acceptable risk Netherlands	$1{,}0 \cdot 10^{-5}$ - $1{,}0 \cdot 10^{-6}$	[SCHNEIDER 1996]
Acceptable risk of developing cancer	$1{,}0 \cdot 10^{-6}$	[KELLY 1991]
Collapse of building	$1{,}0 \cdot 10^{-7}$	[RACKWITZ 1998]
De minimis risk for the public	$1 \cdot 10^{-8}$	[PATÉ-CORNELL 1994]

Table 2- 16: Acceptable risk bases

Mortality rates are based on observations and offer a mean value distributed over a predefined value of time. They are uncomplicated in their application and serve easily as a value for the definition of acceptable risks for individuals. What they do not include however, is the time spend performing an activity and relating it to the probability of dying.

2.3.4 *Fatal Accident Rates*

This drawback is overcome by the *Fatal Accident Rate,* or *FAR*. This measure includes the time period spent in performing an activity. The number of deaths per unit population is then standardized to an exposure time of 10^8 hours, which corresponds to approximately 11 415 years. This is done to avoid very low numbers. The process of FAR during a typical daily routine is depicted in figure 2-8 after [KUMAMOTO AND HENLEY 1996].

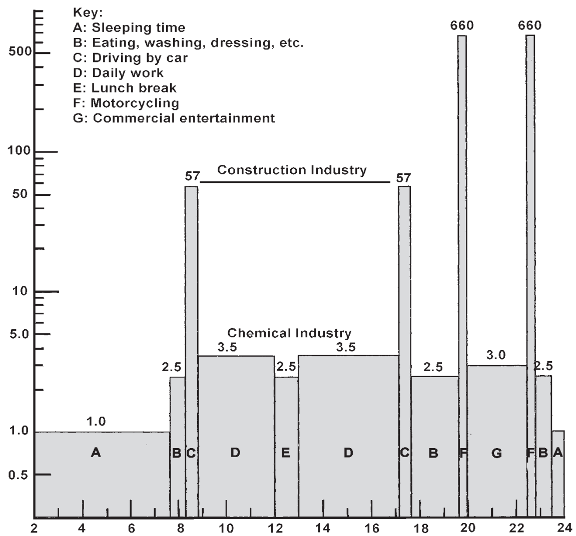


Figure 2 - 8: FAR of daily activities after [KUMAMOTO AND HENLEY 1996]

Action	FAR		Action	FAR
Boxing	7000		Car driving	60
Rock climbing	4000		Coal mining	45
Motorcycle driving	660		Chemical industry worker	3,5
Crew of an airplane	250		Staying at home	3
Driving a bike	96		Driving by bus	3
Construction worker	67		Explosion of a tank	0.0006

Table 2 – 17: Typical FAR for different activities

Other FAR rates from [BEA 1990] are presented in table 2-17. Like the probability of dying, the Fatal Accident Rates are used to determine acceptable levels for individual risk. Which of both is to be preferred is a matter of ongoing discussion.

2.3.5 *Lost Life Expectancy and Years of Life Lost*

All the concepts commented so far did not include the age of the persons dying. While dying at an old age seems natural, the death of younger persons appears more like a tragedy. To take this matter into account, the *Lost Life Expectancy*, or shortly *LLE* was developed by [COHEN 1991], [COHEN 2003]. LLE is easily comprehensible because it relates risks in terms of understandable every day experience. Very basically, the LLE subtracts the difference in the mean life expectancy by the mean age persons of those persons, who die while performing an action or being exposed one. In this way the premature death of an elderly person is influencing the result less than the death of a young person. This is a powerful concept to include the effects of diseases and social influences, as may be seen in figure 2-9. Data marked with an asterisk are those averaged over the entire U.S. population, while the others only include the persons involved in the activity. Thus the data have to be regarded with care.

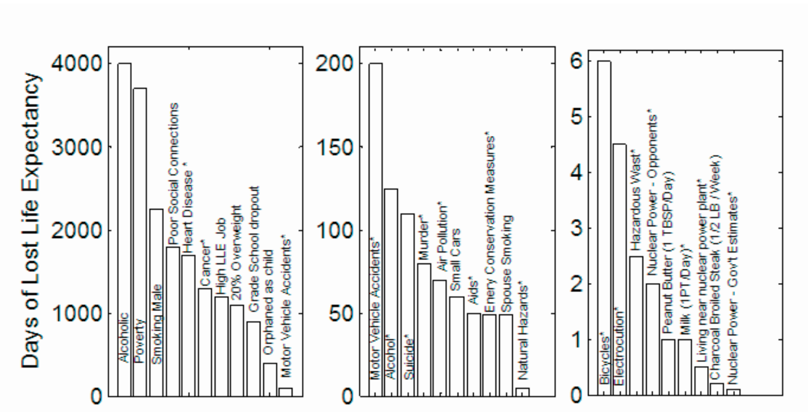


Figure 2 - 9: Overview of LLE after [COHEN 2003]

The concept of LLE was expanded by [VISCUSI ET AL. 1997] who introduced the *Years of Life Lost*, or *YLL*. It is essentially similar to LLE; however, it measures the average expected years of life lost per member of the population, rather than the average expected years of life lost per victim, who was performing or was being exposed to an action. In this way, the data of Cohen may be revised more efficiently. Also, conditions which lead to some loss of the quality of life, such as diseases or critical accidents, may be included by the so called *Quality Adjusted Life Years (QALY)*, the *Disability Adjusted Life Years (DALY)* and the *Health Years Equivalent (HYE)*. A detailed discussion is beyond the scope of this study, but the interested reader is referred to [HOFSTETTER AND HAMMIT 2001] for more information. For this study, they are not of any relevance.

The author feels very sensible towards this kind of comparing risks, since it assigns a higher value to younger persons. This might be justified if deaths as a result of health defects are related to transport risks, but it is unjustified if the death of a 40 year old person as a consequence of a sport accident is related to the death of a 18 year old young person in a traffic accident.

2.3.6 Probability-Damage diagrams

2.3.6.1 Introduction

The measures previously introduced were only related to the assessment of an individual risk. The assessment of societal risks must regard further conditions. One of the important aspects of societal risk is the already mentioned influence of the individual risk perception. The severity of an extreme event may for example be expressed by a risk conversion factor up to 100, as it was articulated in table 2-10. To account for these effects and especially to include the societal issues of determining acceptable risk bases, *Probability-Damage* or *P-D diagrams* were developed. Sometimes they are also referred to as *F-N diagrams*, where F is the frequency and N the number of fatalities. The origins go back until 1967 to the assessment of societal risks of nuclear power stations. P-D diagrams plot the consequences of extreme events, e.g. the losses, versus their probability in a logarithmic scale. Mostly, the loss corresponds to the lives lost, but in general all other measures are applicable as well, such as economic or ecological damage. The reference time is normally taken to be a year, but should also be indicated on the vertical axis. In figure 2-10 on the right some sample results are shown. The picture is based on information given in [SHIP 2001], which in turn cites [WHITMAN 1984]. Additional information may be found in [BEA 1990]. As can be seen, they are extremely well suited for the comparison of different risks. In this diagram, two lines are shown, which are labelled *marginally accepted* and *accepted*.

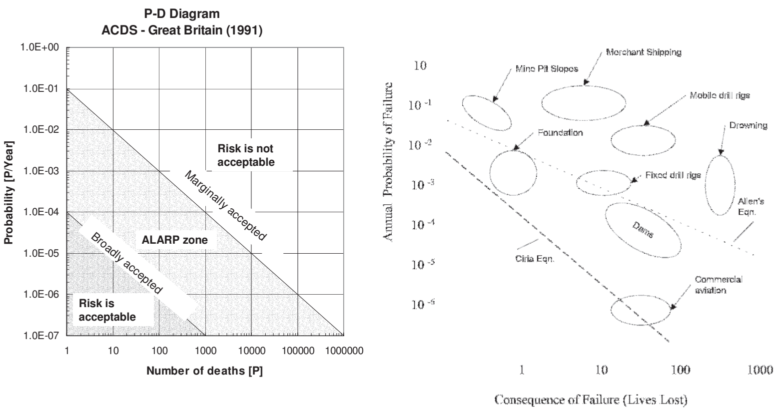


Figure 2 - 10: Probability-Damage diagrams

This differentiation indicates that it is a complicated task to grade whether a risk is acceptable or not. Numerous variables affect the process. Thus, those two lines are widely applied in practice, dividing the diagram in three parts. The first is the *unacceptable* region, where the risk is not to be tolerated. On the other side of the spectrum is the region where it can be said that the risk is *broadly acceptable*. In between those two, the *tolerable* region lies.

With reference to the background risk of approximately one hundredth per year of dying averaged over the lifetime, the level of 10^{-6} is commonly referred to as broadly acceptable [HSE 2001], [KELLY 1991]. The borderline between the broadly acceptable and the tolerable region is set to be the curve at which the cost of reduction would exceed the improvement gained. The cost to be evaluated includes the assessment of the value of human life. The valuation of human life is shortly highlighted in chapters 2.3.7 and 2.3.8. It is difficult to find a similar widely applicable boundary between tolerable and unacceptable risk. [HSE 2001] proposes 10^{-3}/a for workers and 10^{-4}/a where imposed on the public as measures for individually acceptable risks. The position of this line is fixed for each problem at those points, where a risk reduction is impracticable or if its cost is grossly disproportionate to the improvement gained. In this way, the tolerable region is subject to constant change. Very often, this region is labelled *ALARP* zone. ALARP is an abbreviation, which is translated differently: *as low as reasonably practicable* [HSE 2001], *as low as reasonably possible* [PROSKE 2004]. Sometimes it is also translated into *as low as reasons permit* [ALLEN 1991]. Similar procedures are *SFAIRP - so far as is reasonably practicable* - or *ALARA - as low as reasonably achievable* [KUMAMOTO AND HENLEY 1996]. In the following, this work will refer only to ALARP, which will be translated according to the HSE definition. Historically, the earliest hint of this concept is given in a court decision of 1949, which is also referenced to and explained in [HSE 2001], the Edwards vs. the National Coal Board:

> "This case established that a computation must be made in which the quantum of risk is placed on one scale and the sacrifice, whether in money, time or trouble, involved in the measures necessary to avert the risk is placed on the other; and that, if it be shown that there is a gross disproportion between them, the risk being insignificant in relation to the sacrifice, the person upon whom the duty is laid discharges the burden of proving that compliance was not reasonably practicable."

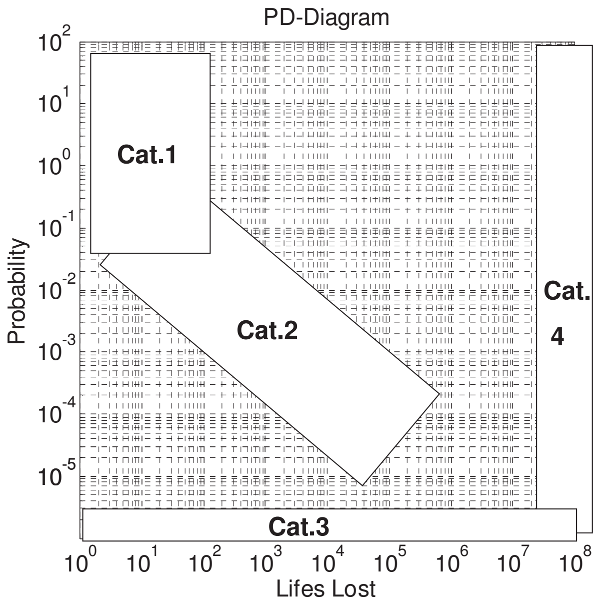


Figure 2 - 11: The different zones of a P-D diagram and the according types of risks modified after [PROSKE 2004].

The different types of risks can be broadly divided into four different kinds, as shown in figure 2-11. The events falling within category 1 are happening often enough so that it is possible to characterize them statistically. Usually those risks do not occur as the result of large consequences. Car accidents are a simple example for this kind of events. Large natural or technical catastrophes fall within category 2. In contrast to [PROSKE 2004], the box of category 2 is inclined. Otherwise the consequences of the event would not dependent on the probability, i.e. the box would be horizontal. Two other categories are explained, although no example may be given. Category 3 events are assumed to be possible, but the probabilities are very low, often below 10^{-8} or less. Although they are possible to calculate, the uncertainty of the result would be too high to provide a reliable result. Finally, the severity of category 4 events comes close to the extinction of mankind or at least full countries. Generally, it may be said that in the left part of the diagram the risks are more individual while as the results move to the right, the impact on the society increases and the area of large public risks is reached.

2.3.6.2 Target values

P-D diagrams are a powerful tool for the assessment and comparison of risks and will be applied in this thesis. Since the determination of acceptable risks plays a large role, the creation of target values for marginally acceptable and acceptable borderlines will be explained. The relation between the probability and the damage is usually expressed by the following formula.

$$P \cdot D^a = B \tag{2.4}$$

Where P is the probability, D the damage, a is the factor showing the risk aversion, which is usually between 1, which corresponds to a constant risk and a value of 2, for which the steepness increases. B is a factor, which moves the line up and down vertically. If the P-D diagram is applied to a single facility, the following expression was derived by VROM, i.e. the Dutch Ministry of Housing, Land Use Planning and Environment [VRIJLING ET AL. 1995]. It is defined as the *locally acceptable level of risk.*

$$1 - F_{N_{dij}}(x) < \frac{C_i}{x^n} \tag{2.5}$$

With the following coefficients:

$F_{N_{dij}}$	=	CDF for the number of deaths resulting from activity i in place j in one year.
C_i	=	A constant determining the position of the limit line, Equivalent to factor *B* in formula 2.3.
n	=	Equivalent to factor *a* in formula 2.3. It determines the steepness of the limit line. A standard line with n=1 is called risk neutral and a steepness of n=2 is called risk averse, since larger accidents are weighed more heavily and consequently only accepted with a lower probability.
x	=	The damage/consequences.

The factor C_i is setting the vertical position of the line. It may be calculated by using formula 2.6, which is derived in [VRIJLING ET AL. 2001].

$$C_i = \left(\frac{\beta_i \cdot Nat_{size}}{k \cdot \sqrt{N_{a_i}}} \right)^2 \tag{2.6}$$

Where the coefficients have the following meaning:

N_{ai}	=	The number of independent places where the event may happen.
β_i	=	A policy factor representing the degree of voluntariness and direct benefit involved. It varies generally between the values of 10 for absolute freedom to the value of 0.01 for imposed risks without a clearly defined direct benefit.
k	=	Constant of trust, which is usually taken as 3.
Nat_{size}	=	Factor to regard the size of the nation in which the risk assessment is performed. It is determined by formula 2.7.

$$Nat_{size} = 7 \cdot 10^{-6} \cdot national \quad population \quad size \tag{2.7}$$

Background of the policy factor β originates in the description of the *individual risk* which is governed by the formula 2.7 [JONKMAN ET AL. 2003]

$$IR = P_f \cdot P_{d|f} \tag{2.8}$$

Where IR is the individual risk, P_f is the probability of failure and $P_{d|f}$ the probability of dying of an individual in the case of failure, assuming the permanent unprotected presence of the individual. The IR is consequently a property of the place; it might also be expressed in dependence of the policy factor as is written in formula 2.9. Corresponding values of the policy factor are described in figure 2-12.

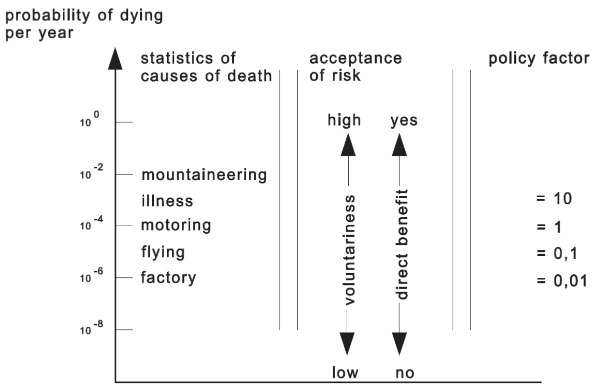


Figure 2 - 12: Policy factor for different activities after [JONKMAN ET AL. 2003]

$$IR \le \beta \cdot 10^{-4} \tag{2.9}$$

Combining these two factors the *personally acceptable level* of risk may be defined as written in formula 2.10, which may also be applied to determine the starting point of the P-D diagram.

$$P_{fi} \le \frac{\beta_i \cdot 10^{-4}}{P_{d|fi}} \tag{2.10}$$

Within the acceptable risks introduced in literature, we also find the *nationally acceptable level of risk* [VRIJLING ET AL. 1995], which is defined in terms of the total risk calculated by the subsequent formula.

$$TR = E(N_{di}) + k\sigma(N) \tag{2.11}$$

With	TR	=	Acceptable total risk
	$E(N_{di})$	=	Expected number of deaths
	k	=	Risk aversion index, similar to the constant of trust in formula 2.6
	σ(N)	=	Standard deviation of the number of deaths

The expected number of deaths is proposed to be solved by multiplying the components:

$$E(N_{di}) = N_a \cdot P_f \cdot P_{d|i} \cdot N_{pi} \tag{2.12}$$

Where	N_a	=	Number of locations regarded
	P_f	=	Probability of failure
	P_{dli}	=	Probability of present at the time of failure
	N_{pi}	=	Size of the population in the regarded location

According to [VRIJLING ET AL. 1995], the total risk should be below:

$$TR \le \beta_i \cdot Nat_{size} \tag{2.13}$$

[VRIJLING ET AL. 1995] applied this approach to some sample applications in the Netherlands for which the factor Nat_{size} is 100. This value is used in literature quite often although applied to other countries. With all these data, P-D diagrams are able to sufficiently describe all relevant measures on all important scales. They were described thoroughly since they will be an important part of this thesis in chapter 6.

2.3.7 *Life Quality Index*

Another important measure in risk analysis is the *Life Quality Index (LQI)*. This parameter, of which several subdefinitions exist, calculates the changes in quality of life by several coefficients. The specifications depend largely on the type of application and may be roughly subdivided into economic, social, medical and engineering life quality factors. Economic life quality takes into account productivity, life expectancy and income. Medical life quality is assessed roughly by means of a questionnaire and applied for different kinds of diseases and injuries. For this study, medical life quality indices are not applicable. A first introduction is offered by [FREI 2003]. Social Life Quality may be expressed by the *Human Development Index* of the UN [UNDP 1990]. This index considers the life expectancy, the ability to write and read and the logarithm of the per capita income. One large and comprehensive study about 22 different life quality indices is given by [HAGERTY ET AL. 2001]. Regarding the topic of this thesis, the certainly most important parameter is the engineering life quality. Originating from [NATHWANI ET AL. 1997], it was promoted in Europe especially by [RACKWITZ 2004], who applied the LQI to the optimisation of acceptable risk levels for technical facilities. This engineering life quality is determined by a function of basically three parameters. The full derivation is given in [NATHWANI ET AL. 1997]. In brief, the main formula is given as follows:

$$L = g^{\frac{w}{1-w}} \cdot e \tag{2.14}$$

Where	g	=	Gross national product
	w	=	Time spend in paid work, used as a life quality measure
	e	=	Mean life expectancy

This means that if money is invested in safety measures, the gross national product decreases while the mean life expectancy increases. In application, this leads to an optimization process of the changes in quality of life as expressed in formula 2.15 and 2.16.

$$\frac{dL}{L} = -C_F \cdot \frac{dM}{M} + \frac{w}{1-w} \cdot \left(1 - \left(1 + \frac{\Delta e}{e}\right)^{1-\frac{1}{w}}\right) \geq 0 \tag{2.15}$$

$$dM = \frac{N_F}{N} \tag{2.16}$$

Where	dM	=	Change in mortality rate
	M	=	Mean mortality rate in the given country
	Δe	=	Change in life expectancy
	e	=	Life expectancy
	N_f	=	Number of possible victims for the given event
	N	=	Number of all victims
	C_f	=	Factor for the population pyramid

In this context, [PROSKE 2004] calculates the maximum costs of a safety measure in dependence of the failure probabilities P_{f1} before the execution and Pf2 afterwards:

$$C = \frac{1-w}{w} \cdot \frac{C_F \cdot N_F}{M} \cdot g \cdot (P_{f1} - P_{f2}) \tag{2.17}$$

Summarized, it can be said that the life quality index generally may not serve as a tool to determine acceptable levels of risks, but enables decision makers to decide between different actions on a nationwide background if risks have been judged to be not acceptable.

2.3.8 The value of a human life

In evaluating the effectiveness of safety instruments, it is often necessary to assign some measure to the value of a human life. This is usually not done by assigning a monetary value to the human life itself. Instead, the most frequent approach followed is to evaluate the willingness to invest into safety measures. These can in turn be expressed pecuniary. Additionally, the measure of human life is often referenced to the average contribution of an anonymous member of the society towards the gross domestic product of the nation. Similar to the LQI, these instruments are used to compare the benefits of several alternatives of safety measures. To round of this chapter and with respect to the larger context of risk management this work is embedded into, short reference is given to some important and commonly used parameters.

Within the context of the LQI, [RACKWITZ 2004] introduced the *Societal Life Saving Cost (SLSC)*, the *Societal Value of a Statistical Life (SVSL)* and the *Willingness to Pay (WTP)*. All values would lead to comparable results, but are based on different derivations of the life quality index. Of those, the most referenced one is the SVSL given in the proximate formula:

$$SVSL = \frac{g}{\frac{w}{1-w}} \cdot E_{aa} \tag{2.18}$$

With g and w being defined before and E_{aa} the age-averaged, discounted life expectancy.

Another approach tries to determine the level, at which the costs outweigh the benefits. Since this value is used for costs related to averting fatalities, it is defined as the *Implied Costs of Averting a Fatality (ICAF)*. Generally speaking, it is the change in costs divided by the change in risk. [KRISTIANSEN AND SOMA 2001] relate it with the LQI and determine it to be:

$$ICAF = \frac{\gamma \cdot e}{4} \cdot \frac{1-w}{w} \tag{2.19}$$

All parameters have been defined before apart from γ being the gross domestic product per person per year. The ICAF value for a well developed country is around three million euro, dated 2001. Detailed examples are given in [SKJONG 2002]. This means that any safety measures with the potential of averting one fatality and total costs below that value should be implemented. Finally, [SKJONG 2002] introduces the *Potential Loss of Life (PLL)*. This parameter is used to determine optimal financial investments for the case study of commercial ships and to transfer outcome of the calculation in such a way that they may be presented in a P-D diagram.

2.4 Concluding remarks

Risk assessment is nothing like a precise measuring of data. Many of the parameters explained before rely on statistics. As for every statistic the background has to be regarded carefully, if not questioned. Results may differ significantly, especially for single facilities, depending on the population size which is included in the study. If risks are assessed, it is also important to realize, who is interested in the assessment of the risk and who is offering the information. Views toward risks may change, whether the same risk is to be seen from viewpoints of an individual, a population, public, agencies or a company. Additionally, the communication of risks in probabilistic terms is often not possible for the general public. Usually verbal expressions like catastrophic, severe, minor, negligible, rare, possible, frequent, et cetera are used. Acceptable risks depend on changes in the regulatory environment as well as in the preferences, values and expectations of the society [HSE 2001]. This source summarizes some of the advantages and drawbacks of several criteria which are used for the determination of acceptable risks.

Equity based	All individuals have the unconditional right to a certain level of protection. This leads to a fixed level of risk. This criterion requires decisions based on a worst case scenario, which resembles least to the reality. Furthermore, this approach leads to uneconomic solutions, since risks are overestimated.
Utility based	Comparison between benefits of measures to reduce risks. Normally this is done by relating monetary terms to relevant benefits, such as years of life lost or statistical lives saved. Utility based criteria tend to ignore ethical or environmental, social, etc. regulations, since they focus often on monetary terms.
Technology based	A satisfactory level of risk is attained, when the “state of the art” measures are employed. Some of these criteria ignore the balance between costs and benefits and thus might lead to inefficient costs, despite reaching an acceptable level of risk.

Table 2 – 18: Overview of criteria to establish acceptable risks according to [HSE 2001].

2.5 Implications for the chosen approach

The comments on catastrophe occurrence showed the importance of the earthquake risk assessment of historical structures. Also, the detailed explanation of different risk measures helps to consider the results in the appropriate surrounding, with sufficient background information about the creation of acceptable risks. Of all the parameters introduced, only the P-D diagrams were able to reflect individual and societal risk, both on a local and a national level. Additionally, great potential is seen in the P-D diagrams since the damage can be expressed in numerous ways. Furthermore, consequences could be articulated by any of the risk measures explained above. The LQI and other definitions to calculate the effectiveness of safety measures were introduced, but for the purpose of this study they are not of primary interest. Thus, the goal must be to determine the consequences of earthquakes in such a way that it will be possible to implement the results in P-D diagrams.

To do so, it is helpful to review the explanations given in chapter 2.1. At first, it is necessary to perform the hazard assessment and to use the results to determine the structural reaction in dependence of the hazard intensity at the given site. Finally, the possible consequences of the structural reaction have to be evaluated. It will be necessary to present results in probabilistic terms, which is in good coherence with the formula for seismic risk assessment proposed by [LEE AND MOSALAM 2006]. The measures show that it is of utmost importance to assess the loss of human life in a detailed manner. This is necessary for a comparison of seismic risks to the diverse kinds of risk explained in this chapter. Another important aspect is that all measures have in common that they include, in some way or another, the amount of human lifetime lost. Thus, it will be necessary for practical purposes to propose at least a rough method to convert CSH values into possible losses of human life, so that they may be implemented in the risk comparison and risk grading.

Chapter 3

Hazard assessment

3.1 Problem statement

Within the seismic risk assessment, the description of the hazard occurrence is the first step to perform. The description of the seismic hazard considerably depends on the way it is used as an input for the vulnerability analysis. Whereas in codes and in the design of buildings deterministic spectral accelerations based on predefined return periods are used, the risk assessment requires a more accurate look at the hazard.

Earthquakes all over the world are characterized by their randomness. It is not only the event itself which is uncertain, but also the main parameters describing the event. Numerous intensity parameters exist that intend to describe the severity of an earthquake. First of all, those parameters have to be determined, which are most closely correlated to the damage occurring in the structure. This is a complicated matter, especially with respect to the duration. Afterwards, the probabilities of exceedance of the governing variables have to be predicted. This cannot be done merely with existing data. As a substitute, artificial accelerograms based on empirical attenuation functions will be used. This chapter will, after a short description of intensity parameters, focus on the topics mentioned above. Moreover, it will address important supplementary topics, such as the different types of uncertainty, the application of natural and artificial accelerograms and the definition of upper bounds for ground motions. Finally, the chapter will conclude with the evaluation of the annual probabilities of exceedance of major intensity parameters, calculated with the programs developed and attached in appendix B.

3.2 Describing and characterizing earthquakes

3.2.1 Introduction

The following text will apply the expressions shown in figure 3-1. They are commonly applied in seismic engineering and are not explained further. Short reference shall also be given to the *source-pathway-receptor* model, which divides the hazard into the three components of source, pathway and site effects. Source effects include for example the hypocentre, fault plane and rupture area, i.e. all mechanisms occurring directly at the fault itself. Next, the pathway determines the medium and the distance through which the event propagates, reflecting the influence of subsoil and ground conditions. Finally, the receptor includes all local site effects as well as parameters describing the exposure and the value of the structure. For this study, the effects at a given site are analysed. Thus, the term hazard always describes the effects at a site and not the parameters at the location of the hazard source. In the following, the major parameters describing the physical characteristics of an earthquake will be explained.

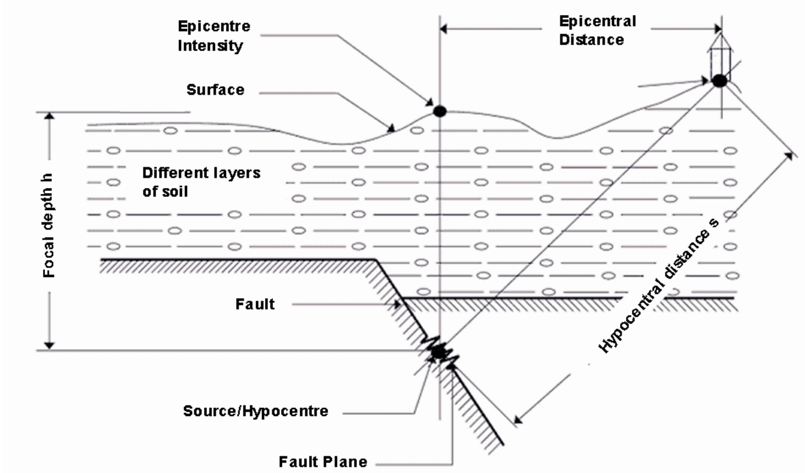


Figure 3 - 1: Definitions in seismic engineering

3.2.2 Global measures

The parameter most closely related to the fault itself is the *seismic moment M_0* representing the physical strength of an earthquake:

$$M_0 = \mu \cdot A_f \cdot D \tag{3.1}$$

Where μ is the shear modulus near the rupture area, A_f the area of the fault and D the average final slip over the fault plane. M_0 is closely correlated to the magnitude, which is commonly referred to in the media and in scientific publications. Still, one has to differ between the different types of magnitudes, which are often not clearly identified. Media usually characterize earthquake events by the *Richter magnitude*, which in scientific publications usually is referred to as *local magnitude M_L*:

$$M_L = \log_{10} A - \log_{10} A_0 \tag{3.2}$$

With A being the maximum recorded amplitude and A_0 a standard value as a function of distance for distances below 600 kilometres. The Richter Magnitude has some drawbacks. It is not related directly to physical characteristics of the earthquake's source and it was originally developed for distances below 600 kilometres and only for a special type of seismometer. Thus two new types of magnitudes evolved – the *body wave magnitude m_b* and the *surface wave magnitude M_s*.

$$m_b = \log_{10}(A/T) + Q_{(D,h)} \tag{3.3}$$

$$M_S = \log_{10}(A/T) + 1.66 \cdot \log_{10} D + 3.3 \tag{3.4}$$

Where A is the maximum amplitude, T the period in seconds, $Q_{(D,h)}$ is a correction factor depending on the distance D in geocentric degrees and the focal depth h. Finally, two more magnitudes shall be addressed, the *moment magnitude* M_w and the *energy magnitude* M_E:

$$M_W = \frac{2}{3} \cdot \log_{10}\left(\frac{M_0}{N \cdot m}\right) - 10.7 \tag{3.5}$$

$$M_E = \frac{2}{3} \cdot \log_{10} E - 2.9 \tag{3.6}$$

If the moment magnitude is evaluated by using formula 3.5, the seismic moment has to be provided and divided by units of Newton meters. The same is true for the units of energy E in formula 3.6. Energy and seismic moment have a different physical background. The energy E radiated by an earthquake is estimated from the spectral energy density of the broadband P-waves. On the other hand, the seismic moment is a measure for the area ruptured by an earthquake. Also, it must be stressed out that the energy used in 3.6 is not the same as calculated by the commonly known formula 3.7 to evaluate the increase in energy as the magnitude increases, cf. also formula 3.31:

$$\log_{10} E = 11.8 + 1.5 \cdot M_s \tag{3.7}$$

A comparison between the different magnitudes may be found in [IDRISS AND ARCHULETTA 2007]. They are not related to each other and differ from site to site, but within the range of magnitude between 4.5 and 7.5, which is of interest for this study, they can be assumed to be similar. Figure 3-2 presents additional results for a comparison of magnitudes.

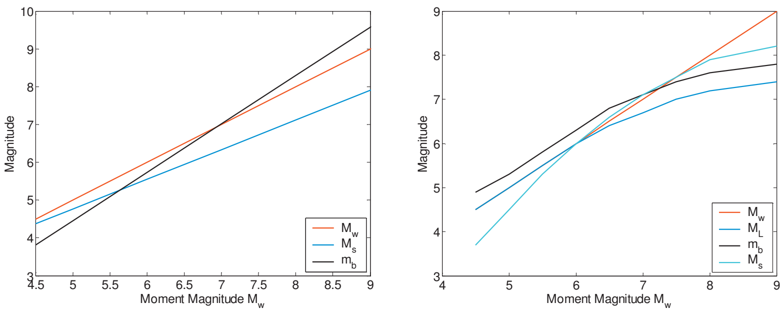


Figure 3 - 2: Relation between moment magnitude and various other magnitude scales after [IDRISS AND ARCHULETTA 2007] on the right and [SUCKALE ET AL. 2005] on the left.

Magnitude must not be confused with *intensity*. Intensity is often employed in seismic engineering especially if dealt with regions of low seismicity. This is because the verbal description of earthquakes allows the analysis of historical sources. The seismic intensity scale is not determined from the physical parameters, but rather from the damage caused to structures or the behaviour of objects within a building and the way it is perceived by human beings. Different intensity scales exist of which in Europe the most common are the *Modified Mercalli Intensity Scale (MMI)* [WOOD AND NEUMANN 1931] and the *European Macroseismic Scale (EMS-98)* [GRÜNTHAL 1998], which is used in this study and thus attached in appendix C.

3.2.3 Time domain

It was already announced that a study will be performed to determine which intensity parameters are well correlated to structural damage. Therefore, some intensity parameters have to be explained. The best known examples are the *peak ground acceleration (PGA)*, the *peak ground velocity (PGV)* and the *peak ground deformation (PGD)*. Another important value frequently used is the *Arias Intensity*:

$$AI = \frac{\pi}{2 \cdot g} \cdot \int_0^{T_0} a_g^2(t)dt \tag{3.8}$$

The factor g is the acceleration of gravity, a_g the ground acceleration and T_0 the total duration of the record. It will be shown that this measure is very well correlated with the structural damage because it takes into account the intensity of acceleration over a period of time. Another measure that will be used in this research was proposed by [BENEDETTI ET AL. 2001] and called the *referential energy r(t)*.

$$r(t) = M \cdot \int_0^t \left| \ddot{u} \cdot \dot{u} \right| dt \tag{3.9}$$

With M being the total mass of the structure and u being the acceleration and velocity respectively. Two other parameters are presented in [COSENZA AND MANFREDI 2000]. The first is the *damage factor* I_D, the second the *Saragoni factor* P_D given with reference to [SARAGONI 1990]:

$$I_D = \frac{2g}{\pi} \cdot \frac{AI}{PGA \cdot PGV} \tag{3.10}$$

$$P_D = \frac{AI}{\nu_0} \tag{3.11}$$

Where ν_0 is the number of zero crossings of the acceleration record in the time history. Although the ground acceleration is mostly used to describe the severity of an earthquake, researchers assign a prominent role to the velocity. One of these velocity aligned measures named *energy density* E_d is defined by Sarma [SARMA 1971]:

$$E_D = \int_0^t v^2(t)dt \tag{3.12}$$

In this case v denotes the velocity. Rather seldom used are *root mean square (RMS)* of the acceleration and the *cumulative absolute velocity (CAV)* using T as total time and a as acceleration of the earthquake:

$$RMS = \sqrt{\frac{1}{T} \cdot \int_0^t a^2 dt} \tag{3.13}$$

$$CAV = \int_0^t \left| a(t) \right| dt \tag{3.14}$$

Finally, some intensity measures also try to include the time such as the *damage factor* J_I used [FAFJAR ET AL. 1990] or the *characteristic intensity* I_c of the earthquake defined as [PARK AND ANG 1985]:

$$J_I = PGV \cdot D_s^{0.25} \tag{3.15}$$

$$I_c = a_{rms}^{1.5} \cdot D_s^{0.5} \tag{3.16}$$

Where a_{rms} is the root mean square of the acceleration and D_s is the duration between the 5% and 75%, respectively 95% of the Husid diagram, which is explained in the subsequent figure. This figure shows the progress of the Arias Intensity for an earthquake on the left. On the right the Husid diagram is plotted, which is the Arias Intensity normalized to the value of 1.

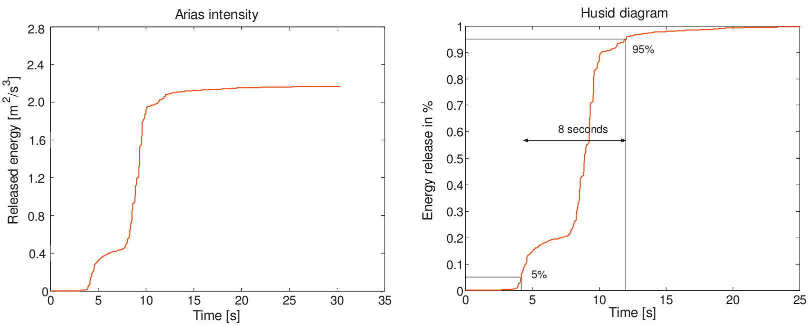


Figure 3 - 3: Arias and Husid plot of an earthquake

A matter still discussed intensively is the impact of the duration of an earthquake. While intuitively one might argue that the duration should have a considerable effect on the damage, the general scientific proof of its importance is to some degree still missing. The debate is somewhat confused since several measures for the time duration are used in literature and science.

Duration may be grouped into three different categories. At first, the *significant duration* D_{s95} is applied regularly and was already defined as the time between 5% and 95% of the Arias Intensity. Sometimes the value referred to is D_{s75}, including only the time between 5% and 75%. The next type of time, the *bracketed duration* $D_{b0.05}$, is defined as the time interval between the first and last exceedance of a given threshold, here taken as 0.05g. Related to the latter measure is the *uniform duration* D_u, which is defined as the sum of time intervals in which the threshold is exceeded. At last, the number of cycles the acceleration crosses the zero point, NC, is used only infrequently.

3.2.4 Frequency domain

Within the frequency domain all measures are related to the frequency content of a given time history. *Response spectra*, *power spectra* and *spectral acceleration* will not be explained further. It is important to keep in mind that response spectra are developed by determining the maximum relative top displacement S_d of a single degree of freedom system with viscous damping. The often cited equation of motion is solved using the Duhamel integral resulting in the following expression determining the *spectral displacement* S_d.

$$u(t) = -\frac{1}{\omega_D}\int_0^t \ddot{u}(\tau)e^{-\xi\omega_n(t-\tau)}\sin[\omega_D(t-\tau)]d\tau \tag{3.17}$$

Now, since the integral term has the unit of m/s, it may be seen as a velocity. This value is also called *pseudovelocity* S_v and may be determined as follows:

$$S_V = \int_0^t \ddot{u}(\tau)e^{-\xi\omega_n(t-\tau)}\sin[\omega_D(t-\tau)]d\tau \tag{3.18}$$

Using 3.17 and 3.18 and including the *pseudoacceleration* S_a a relationship between the three components may be derived:

$$S_a = \omega_D{}^2 S_D = \omega_D S_V \tag{3.19}$$

The pseudovelocity-spectrum is not the same as the absolute velocity spectrum. A full solution for the velocity would result in.

$$\dot{u}(t) = -\xi\omega_n u(t) - \int_0^t \ddot{u}(\tau)e^{-\xi\omega_n(t-\tau)}\cos[\omega_D(t-\tau)]d\tau \neq \omega_D\ S_D \tag{3.20}$$

Nevertheless, pseudo-values are commonly used and may easily be applied in combined S_d, S_v, S_a Spectra as it is for example done in figure 3-5. Eventually, the last important measure to know is the *spectral intensity SI*, which is the area below the pseudovelocity spectrum in the period range from 0.1 second to 2.5 second. It is a measure supposed to be strongly correlated with the structural damage [VON THUN ET AL. 1988].

$$SI(D) = \int_{T=0.1}^{2.5} S_v(T,D)dT \tag{3.21}$$

Instead of integrating the pseudovelocity, some applications prefer integrating the pseudoacceleration spectrum; in this case the result is labelled *acceleration spectrum intensity*, or *ASI* [VON THUN ET AL. 1988]. Periods between which the spectra are integrated usually range from 0.1 second to 0.5 seconds.

$$ASI = \int_{T_0}^{T_1} S_a(T,D)dT \tag{3.22}$$

Very rarely the *mean period* of an earthquake is applied, defined after [RATHJE ET AL. 1998] as follows:

$$MP = \frac{\sum C_i^2 / f_i}{C_i^2} \tag{3.23}$$

Where MP is the mean period of an earthquake, $C_{i,}$ the Fourier amplitudes and f_i the discrete Fourier transformed frequencies between 0.25 Hz and 20 Hz.

3.3 Natural and artificial earthquake records

3.3.1 Natural records

It was already described before that the prediction of characteristics and different levels of a ground motion is the major task to be performed in the assessment of seismic hazard. Recordings of natural earthquakes are available online at several databases, e.g. [PEER 2006] and [ESD 2006]. Still, those records are not always of the magnitude, distance or soil conditions which are to be considered in the structural analysis. Additionally, it is not always traceable, whether – and if, how – the records are filtered or modulated. Since for this work a large number of different records are going to be needed, the approaches for the applied generations of artificial accelerograms shall be explained shortly.

3.3.2 Artificial records

3.3.2.1 Random phase angle

For the generation of artificial accelerograms several algorithms exist. In general, it may be said that they all base on a given response or power spectrum and are created by an inverse Fourier transformation which is then modulated, filtered and iteratively fitted to the given spectrum. For application in risk assessment, it has to be evaluated which algorithm is the most suitable. The easiest and probably most common way in the generation of artificial accelerograms is the summation of different harmonic waves with randomly varying phase angle. To do so, a linear elastic response spectrum has to be presumed at first. From this, the earthquake time history is developed. In this approach, the duration of the record has to be estimated or based on observed data since this information is not available in the design spectrum. Finally, the generated record is multiplied with an intensity function. The approach is already implemented in programmed codes such as [MESKOURIS 1999], which is used in this study if reference is made to random phase angle generation.

Since the adopted response spectrum of the (pseudo-) acceleration is ruling the generation and input of this function it deserves some extra attention. Generally, the design spectra given in the codes will be applied. This procedure is without doubt the best if a new building has to be designed. For the assessment of the risk of an existing building this approach contains some drawbacks. At first, the generated earthquakes represent a very high energy, since all frequencies in the design spectrum are included in the generated record. Secondly, the critical task of the duration is not addressed. Moreover, no information relating the magnitude, the distance or the probability is given with reference to response spectra, which will be important for the course of this study. Also, it is not always clear, what exceedance probability is underlying the code design spectra. In Germany research was conducted to overcome some of these drawbacks. The discussion arose about the application of design spectra developed for – in this case two – given locations of high risk nuclear power stations. Most of the work and the ideas are based on the work of [HOSSER 1987], who developed design spectra based on the statistical evaluation of freefield spectra for three different intensity classes and also depending on the ground conditions. Results of this research reverberate in discussion [RSK 2002], [RSK 2004]. This discussion is about which occurrence probability and what type of response spectra should be used as a reference for the safety assessment of these structures. It can be said that the logarithmized spectral amplitudes are normal distributed. Thus, the mean of the logarithm is equal to the 50% quantile of the spectral acceleration. [HOSSER 1987] also found that for his set of data the standard deviation of the logarithms is approximately equal to the coefficient of variance which is around 60 % for all tests.

Within the reactor safety commission it was discussed, whether the response spectra with a probability of occurrence of 10^{-5} and the 50% quantile should be used, or the 10^{-4} probability of occurrence but including the standard deviation, i.e. the 84% quantile. Most codes use the broadband 84% fractile. This spectrum envelopes the random frequency contents of a larger number of earthquakes resulting in an unnatural broadband spectrum. Additionally, this quantile should be applied only if a larger database was used, since the quantile reacts very sensible towards additional records if only a low number of observations are included. Artificial accelerograms considering this spectrum are reflecting the unnatural broadband spectrum. [HOSSER 1987] points out, that in summation very conservative approaches are implemented. Not only is the probability of occurrence low, but especially design response spectra are scaled with very conservative assumptions of the peak ground motions and added with a strong motion duration assumed to be significantly too long.

Concerning the duration, it may be said that even now many researchers keep in mind the duration of the El Centro earthquake of which some records have been extraordinary long. This duration occurred due-to very special ground conditions, which of course are contributing largely to the hazard but are seldom found in reality. The prediction of the occurrence probability of the duration will be one of the important points of this work.

This is why the spectra given in the codes are not applied in this study; instead response spectra are generated by empirical attenuation functions. They typically consist of some of the parameters shown in formula 3.24. They generally include magnitude M, distance R and depth h. A more thorough description is given in chapter 3.5:

$$\ln Y = C_1 + C_2 \cdot M + C_3 \cdot M^{C_4} + C_5 \cdot \ln\sqrt{R^2 + h^2} + C_6 \cdot \sqrt{R^2 + h^2} + f_{(Source)} + f_{(Soil)} \tag{3.24}$$

Where:

Y	=	Intensity parameter, such as PGA or S_a
C_1	=	Related to the unit of Y
C_2-C_4	=	Exponential relation between magnitude and released energy
C_5	=	Geometrical distribution of energy
C_6	=	Attenuation of the energy
f	=	Functions for fault mechanisms and soil influence

3.3.2.2 Modulated white noise

The second major type of algorithm for the generation of artificial accelerograms is based on the modulation of white noise. The approach considered in this study was presented by [ARMOUTI 2004]. The ground acceleration of this form is expressed by formula 3-25:

$$\ddot{u}_g(t) = PGA \cdot e(t) \cdot X(t) \tag{3.25}$$

Where:

PGA	=	Peak ground acceleration
e(t)	=	Normalized nonstationary envelope function
X(t)	=	Stationary filtered white noise

X(t) may be expressed in the following form:

$$X(t) = \sum \sqrt{2 \cdot S_g(\omega_n) \cdot \Delta\omega} \cdot \cos(\omega_n t + \theta_n) \qquad n = 1,2,3,\ldots m \tag{3.26}$$

Where:

$S_g(\omega_n)$	=	Earthquake one sided power spectral density
ω_n	=	Frequency of the n-th generated sample
m	=	Number of samples
θ_n	=	Random phase angle of the n-th generated sample between 0 and 2π

The shape of the power spectral density function $S_g(\omega_n)$ is created with the Kanai-Tajimi filter, cf. formula 3.26, and depends on the main earthquake frequency and the damping ratio, which are both considered to be only dependent on the type of soil at the considered site.

$$|H(\omega)|^2 = \frac{1 + 4 \cdot \zeta_g^2 \cdot (\omega / \omega_g)}{\left[(1 - (\omega/\omega_g)^2\right]^2 + 4 \cdot \zeta_g^2 (\omega/\omega_g)^2} \tag{3.27}$$

Where:

H	=	Shape of the distribution of the spectrum
ω_g	=	Dominant earthquake frequency
ς	=	Damping ratio of the site

The parameters ω_g and ς of the site influencing the time history are shown in the following table.

Soil Type	ω_g	ς
Rock	8π	0.6
Deep cohesionless	5π	0.6
Soft	2.4π	0.85

Table 3 - 1: Parameters for modulated white noise approach

3.3.2.3 Nonstationary Approach

Simulating the nonstationary characteristics, apart from modulating the acceleration by multiplying them with envelope functions can only hardly be performed. Still, one approach, which is adopted in this thesis, was developed by [SABETTA AND PUGLIESE 1996]. This approach is very consistent, since it starts with the input parameters magnitude, distance and soil type and leads to artificial accelerograms. Next to the summation of Fourier series with random phases, it also includes time dependent coefficients, in this way including nonstationary characteristics. The frequency content is time dependent as well as the amplitude. To do so, they use what is called a physical spectrum. The physical spectrum consists of several power spectral densities at different times. Governing equations are listed in appendix D. A program code for the generation of artificial earthquakes may be found in [LESTUZZI 2007]. For reasons of completeness it shall be added that naturally more algorithms for the generation of earthquake exist. Especially those developed by seismologists certainly deserve attention; cf. sources given in [SABETTA AND PUGLIESE 1996]. Still, they are not dealt with here for they are based on rupture processes, type and dimension of the fault and require knowledge of parameters which are not common in civil engineering and often hard to estimate in advance.

3.3.3 Comparison

Overall, it can be concluded that each algorithm and each way to produce artificial earthquake records may be modulated and fitted to reasonable results. Also, depending on the type of building and the goal why the structural analysis is performed, it cannot be generally concluded, which algorithm might be preferred. Since in modern codes target spectra are given, the approach started by [MESKOURIS 1999] is the easiest applicable without any given problems. Only the duration has to be adjusted to the site.

Parameter	NR 1	NR 2	NR 3	NR 4	Armouti	Sabetta	Meskouris
Maximum Acceleration [m/s²]	1.33	1.60	2.11	1.45	1.3	1.12	1.3
Maximum Velocity [cm/s]	12.84	15.34	25.63	17.18	27.4	16.5	27.2
Maximum Displacement [cm]	1.50	1.76	7.58	3.31	25.68	1.29	3.51
V_{max}/A_{max} [sec]	9.65	9.57	12.1	11.81	21.06	6.05	20.9
Acceleration RMS [m/s²]	0.19	0.206	0.44	0.39	0.32	0.204	0.62
Velocity RMS [cm/s]	1.79	1.81	6.45	5.08	8.7	1.42	10.3
Displacement RMS [cm]	0.309	0.371	2.18	1.07	8.81	0.29	1.23
Arias Intensity [m/s]	0.173	0.207	0.89	0.77	0.65	0.12	0.41
I_c	0.45	0.516	1.56	1.33	1.14	0.383	0.93
E_D [cm²/s]	97.05	99.96	1220	7.46	3032	85	245
CAV [cm/s]	2.88	3.14	7.89	8.2	9.56	2.06	6.87
ASI [cm/s²]	1.23	1.4	2.01	1.5	1.16	1.02	1.42
VSI [cm/s]	51.84	47.61	107	96.6	59.26	29.82	51.84
D_u 0.05g [s]	1.03	1.25	4.45	5.21	5.1	0.61	6.2
D_b 0.05g [s]	6.14	6.36	13.75	16.98	27.97	5.72	9.5
D_s 95 [s]	7.88	6.99	12.96	16.11	25.95	5.86	9.5

Table 3 - 2: Comparison of the properties of natural and artificial accelerograms

The approach is especially useful for determination of the structural safety for an earthquake based on specific response spectra. For practical applications in risk assessment it misses the opportunity to include variabilities in location and magnitude. Also, the energy of real earthquakes is overestimated as short sample studies, whose results are presented in table 3-2, have shown. The same is true for the generation of artificial earthquakes based on the modulation of white noise. In detail, the approach of [ARMOUTI 2004] creates earthquakes of a long duration and a high energy, which in contrast to natural earthquakes is released rather steadily. A numeric comparison of some results for the comparison of natural and artificial recordings was performed for four earthquakes on soft soil and four on stiff soil, which had an epicentral distance of 30 kilometres and a surface magnitude close to 6.0. The results, which show the effects mentioned are plotted in table 3-2 and figure 3-4.

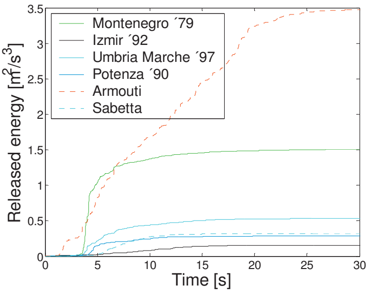


Figure 3 - 4: Comparison of the Arias Intensity of soft soil earthquakes

Due-to the results of the comparison in the preceding chapter, the generation of artificial earthquakes with the algorithm developed by [SABETTA AND PUGLIESE 1996] is clearly favoured. Some sources also recommend to include the impact of the different fault mechanisms [RSK 2002], which is not done here, because [ÓLAFSSON ET AL. 2001] determine magnitude and distance are the most important parameters. Finally, Sabetta's and Pugliese's algorithm is the most sophisticated one in comparison to the other approaches, especially since it provides reliable estimates of duration and includes the nonstationary development of frequencies over time.

3.4 Damage correlated intensity measures

3.4.1 Overview

Intensity measures describe the strength of an earthquake by its physical properties. The most important measures were introduced in chapter 3.2. It is one of the main – but still open – tasks of engineers to determine those, which are most closely correlated with the structural damage. Until today, the task remains unsolved although new parameters are introduced yearly. This is due to a number of different effects, mainly because the impact depends on the type of building and type of the material used for its construction. Also, the description of damage may differ, e.g. local crack width, strength or stiffness degradation. To overcome these difficulties, a numerical study is performed to identify, which intensity parameters are best correlated to the damage evaluated with the applied material model. This will be one of the core tasks of this study. The applied material model was developed by [GAMBAROTTA AND LAGOMARSINO 1997] and further improved by [CALDERINI AND LAGOMARSINO 2004]. The material model is able to describe the post-peak behaviour of the masonry material and the dynamic behaviour by dissipation through frictional mechanisms. It is a damage model in which the inelastic strains are described by means of two internal damage variables that describe the damage evolution in the bricks and in the mortar. A longer explanation is offered in chapter 4.1. Since the model applied is a *damage model*, the hope is that the two internal damage variables will be able to depict the damage in a reliable way. In the end, the results will be compared to a study of [BOMMER ET AL. 2004a] which is at the moment of writing this thesis the most complete overview of several intensity measures and their correlation to structural damage.

3.4.2 Test Calculations

For the study one specific artificial earthquake based on the summation of harmonic components with randomly chosen phase angles was generated and fitted to a response spectrum. The length of this accelerogram was set to eleven seconds with a linearly increasing intensity function during the first second and a decreasing one in the last second. The following figure shows the accelerogram on the left and the pseudovelocity spectra as the black line on the right.

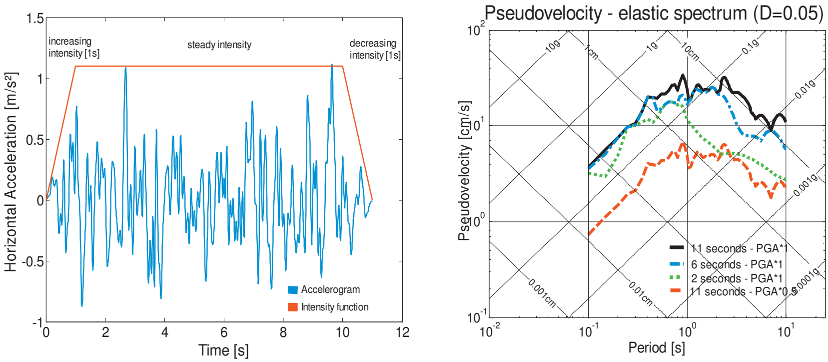


Figure 3 - 5: Obtained accelerogram and pseudovelocity spectra for some of its variations

Now a set of 100 earthquakes was created by multiplying the amplitudes of the "source" earthquake by factors 0.5 to 5.0 in steps of 0.5 and changing the length of the accelerogram from two seconds to 11 seconds in steps of one second. A set of 100 earthquakes was obtained, which was then applied to a finite element model of a wall with changing height, i.e. different first eigenfrequencies ranging from 0.5 Hz to 10 Hz, resulting in a total of 1000 calculations.

Although this approach does not reflect the true variability of ground motions, it is chosen because the impact of PGA and duration can be evaluated separately, without the other intensity parameters changing and influencing the results too much. To evaluate the damage, five structural damage parameters were used. Naturally, both internal damage variables describing the loss of strength in mortar and brick are considered. The loss of strength and the loss of stiffness both depend on these two parameters. Other parameters used as an expression for the structural damage include plastic shearing and plastic vertical strains, the top displacement as well as vertical and shear stresses. Concerning the strains it has to be added that the model describes cracks by large strains, without losing the connection, but by reduction of the resistance with respect to further cracking. This means that if a certain strain is exceeded once, the resistance towards cracking decreases and will decrease further as the strains increase, cf. figure 3-6.

Strains and model inherent damage variables are often localized. Thus both, the localized maximum damage in one element of the numerical model – referred to as local damage in the rest of the text – and the damage accumulated over the structure per element – in the following called global damage – were taken as descriptors of damage. In the following, the abbreviations of table 3-3 will be used.

Abbreviation	**Description**
HD	Horizontal displacement
MD	Localized mortar damage
MDS	Global mortar damage
BD	Localized brick damage
BDS	Global brick damage
CV	Vertical crack
CVS	Integration of vertical crack width over time
CH	Shear cracks
CHS	Integration of shear crack width over time
MS	Maximum equivalent stress
MSH	Maximum shear stress

Table 3 - 3: Abbreviations used in the following text

These results were already presented in some detail in [URBAN ET AL. 2006] and [PEIL AND URBAN 2006]. They will be explained throughout chapter 3.4.3, especially when the influence of duration is assessed, in which this research proved to be very useful.

Still, this work had the drawback that it was based on variations of the same artificial accelerogram. To avoid missing the natural variation of earthquakes, the study was expanded. Therefore 100 time histories were included additionally of which 30 were natural recordings, 10 were developed with the algorithm of [ARMOUTI 2004], 20 by the random phase angle approach presented [MESKOURIS 1999] and 40 more by using the [SABETTA AND PUGLIESE 1996] approach. This was done with the purpose to include events between a Local Magnitude of 4.5 and approximately 7.0 and distances ranging between 10 and 40 kilometres. Again, each time history was applied to walls with different natural frequencies adding 1000 simulations to the database. The results did not alter significantly from the data presented in [URBAN ET AL. 2006]; although the correlations coefficients were slightly lower in the latter case. Thus, the following tables will show the correlations for the full set of 2000 calculations.

Before the results are discussed, additional information about the significance of the two internal damage parameters has to be supplied. A deeper description of the material model implemented in this study is to be found in chapter 4.1.4. So far, it is only necessary to know that the internal damage parameters determine strength and stiffness degradation of the mortar bed joints and the bricks. The strains at the nodes of the numerical finite element model are determined by an elastic and an inelastic contribution. The inelastic part is influenced by the type and direction of local stresses and the evolution of the mortar and brick damage variable. This variable describes the loss of toughness at each node of the element. Loss of toughness might be described as the decrease in energy needed to cause further cracking, expressed as the percentage of the energy needed in the undamaged state. If cracking occurs, the structure may react very brittle or nearly plastic. The softening parameter β characterizes whether the material reacts brittle, in that case β would be equal to one, or ideal plastic for a β of 0. To account for the post-peak behaviour the resistance towards cracking may be described by formula 3.28, which would result in figure 3-6 for different values of β. In the formula α is the damage parameter used in the material model. Throughout studying the influence of intensity parameters a value of β=0.8 for mortar and β=0.4 for the bricks was used. With these explanations in mind, the analysis of the results may be started.

$$R(\alpha) = \begin{cases} R_c \cdot \alpha & 0 < \alpha < 1 \\ R_c \cdot \alpha^{-\beta} & \alpha > 1 \end{cases} \tag{3.28}$$

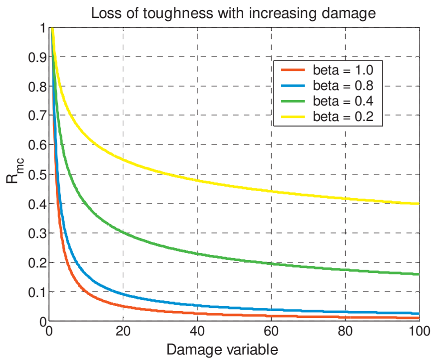


Figure 3 - 6: Loss of toughness

3.4.3 Results

Most parameters showed a nonlinear relationship as may be seen for example in figure 3-7. This is why tables 3-4 to 3-8 present the results of the rank-order correlation coefficient. This correlation coefficient is more suitable than the linear correlation coefficient if nonlinear relationships want to be analysed.

The results will be presented in four steps. At first basic time-domain parameters related to PGA, PGV and PGD are listed in table 3-4. Afterwards, the more sophisticated parameters in the time domain will be assessed followed by those of the frequency domain. Finally, the importance of the different measures for the duration of an earthquake will be stressed out.

	HD	MD	MDS	BD	BDS	CV	CVS	CH	CHS	MS	MSH
10 Hz											
PGA	0.95	0.97	0.96	0.98	0.97	0.98	0.76	0.76	0.94	0.98	0.96
PGV	0.92	0.91	0.91	0.91	0.91	0.92	0.69	0.69	0.93	0.91	0.92
PGD	0.67	0.67	0.67	0.66	0.67	0.68	0.54	0.54	0.68	0.67	0.69
Acc_{RMS}	0.95	0.97	0.97	0.97	0.98	0.98	0.76	0.76	0.96	0.98	0.98
Vel_{RMS}	0.90	0.91	0.90	0.91	0.91	0.92	0.71	0.71	0.92	0.92	0.93
Dis_{RMS}	0.78	0.77	0.77	0.77	0.77	0.78	0.57	0.57	0.79	0.77	0.80
SMA	0.95	0.98	0.97	0.98	0.98	0.98	0.77	0.77	0.97	0.98	0.98
SMV	0.85	0.86	0.87	0.85	0.85	0.87	0.68	0.68	0.88	0.86	0.88
EDA	0.95	0.97	0.97	0.98	0.98	0.98	0.76	0.76	0.96	0.98	0.98
A_{95}	0.93	0.96	0.97	0.97	0.98	0.97	0.75	0.75	0.94	0.97	0.96
5 Hz											
PGA	0.94	0.92	0.93	0.95	0.96	0.95	0.88	0.93	0.88	0.95	0.93
PGV	0.95	0.90	0.93	0.94	0.95	0.95	0.76	0.94	0.82	0.95	0.92
PGD	0.73	0.65	0.68	0.68	0.71	0.69	0.55	0.69	0.64	0.69	0.65
Acc_{RMS}	0.98	0.93	0.96	0.97	0.98	0.97	0.85	0.96	0.88	0.97	0.94
Vel_{RMS}	0.96	0.89	0.93	0.93	0.95	0.95	0.75	0.93	0.84	0.94	0.88
Dis_{RMS}	0.86	0.75	0.84	0.80	0.83	0.81	0.58	0.81	0.69	0.81	0.76
SMA	0.96	0.93	0.95	0.98	0.98	0.97	0.85	0.95	0.88	0.97	0.94
SMV	0.92	0.84	0.89	0.88	0.91	0.89	0.67	0.90	0.80	0.88	0.84
EDA	0.95	0.92	0.94	0.96	0.97	0.97	0.86	0.93	0.87	0.96	0.93
A_{95}	0.93	0.91	0.92	0.94	0.95	0.94	0.88	0.92	0.87	0.95	0.92
2 Hz											
PGA	0.83	0.74	0.84	0.85	0.88	0.82	0.65	0.78	0.77	0.83	0.84
PGV	0.95	0.87	0.97	0.97	0.98	0.96	0.63	0.88	0.85	0.95	0.94
PGD	0.77	0.72	0.77	0.75	0.77	0.77	0.64	0.71	0.68	0.77	0.76
Acc_{RMS}	0.88	0.78	0.89	0.89	0.94	0.87	0.64	0.83	0.81	0.88	0.88
Vel_{RMS}	0.94	0.87	0.94	0.96	0.97	0.95	0.63	0.87	0.85	0.94	0.93
Dis_{RMS}	0.90	0.82	0.88	0.89	0.90	0.87	0.58	0.84	0.79	0.84	0.88
SMA	0.87	0.76	0.88	0.88	0.92	0.86	0.62	0.81	0.79	0.86	0.87
SMV	0.95	0.88	0.97	0.93	0.95	0.92	0.58	0.87	0.84	0.91	0.93
EDA	0.84	0.75	0.86	0.86	0.90	0.83	0.62	0.80	0.78	0.84	0.85
A_{95}	0.81	0.72	0.82	0.83	0.88	0.80	0.64	0.77	0.74	0.82	0.82
1 Hz											
PGA	0.76	0.85	0.76	0.85	0.93	0.86	0.65	0.84	0.83	0.86	0.83
PGV	0.94	0.94	0.88	0.93	0.97	0.94	0.64	0.97	0.84	0.97	0.89
PGD	0.76	0.73	0.62	0.71	0.72	0.73	0.54	0.77	0.67	0.76	0.65
Acc_{RMS}	0.83	0.90	0.81	0.91	0.97	0.92	0.65	0.91	0.84	0.92	0.87
Vel_{RMS}	0.93	0.94	0.82	0.93	0.97	0.94	0.68	0.95	0.85	0.97	0.87
Dis_{RMS}	0.91	0.87	0.77	0.85	0.85	0.87	0.61	0.91	0.76	0.90	0.80
SMA	0.80	0.88	0.80	0.89	0.96	0.89	0.65	0.89	0.85	0.90	0.86
SMV	0.93	0.92	0.87	0.94	0.93	0.93	0.67	0.95	0.84	0.96	0.86
EDA	0.79	0.86	0.78	0.87	0.94	0.87	0.65	0.87	0.83	0.88	0.85
A_{95}	0.75	0.83	0.75	0.84	0.91	0.84	0.64	0.83	0.80	0.85	0.82
0.5 Hz											
PGA	0.57	0.71	0.71	0.73	0.34	0.79	0.45	0.84	0.61	0.81	0.69
PGV	0.80	0.87	0.87	0.88	0.61	0.92	0.33	0.91	0.71	0.92	0.87
PGD	0.72	0.70	0.68	0.69	0.45	0.78	0.29	0.73	0.60	0.71	0.65
Acc_{RMS}	0.65	0.78	0.78	0.80	0.43	0.85	0.44	0.88	0.66	0.86	0.77
Vel_{RMS}	0.78	0.86	0.85	0.87	0.60	0.92	0.37	0.91	0.73	0.90	0.82
Dis_{RMS}	0.83	0.85	0.85	0.89	0.61	0.89	0.26	0.84	0.68	0.84	0.78
SMA	0.62	0.76	0.77	0.79	0.41	0.83	0.42	0.86	0.63	0.84	0.76
SMV	0.79	0.89	0.87	0.87	0.65	0.92	0.31	0.89	0.76	0.90	0.83
EDA	0.59	0.74	0.75	0.76	0.38	0.81	0.43	0.84	0.60	0.82	0.73
A_{95}	0.56	0.68	0.69	0.71	0.32	0.77	0.45	0.81	0.60	0.79	0.68
All frequencies											
PGA	0.57	0.77	0.60	0.60	0.14	0.80	0.62	0.83	0.67	0.72	0.82
PGV	0.64	0.80	0.63	0.64	0.14	0.85	0.56	0.88	0.68	0.77	0.84
PGD	0.49	0.60	0.46	0.47	0.09	0.63	0.45	0.66	0.52	0.57	0.62
Acc_{RMS}	0.60	0.79	0.62	0.63	0.14	0.84	0.61	0.86	0.69	0.75	0.84
Vel_{RMS}	0.63	0.80	0.62	0.63	0.14	0.85	0.57	0.87	0.69	0.76	0.84
Dis_{RMS}	0.58	0.70	0.56	0.56	0.11	0.76	0.45	0.76	0.59	0.68	0.75
SMA	0.59	0.79	0.62	0.62	0.14	0.83	0.60	0.86	0.68	0.74	0.84
SMV	0.62	0.77	0.61	0.61	0.13	0.83	0.54	0.85	0.66	0.74	0.80
EDA	0.58	0.78	0.61	0.61	0.14	0.82	0.60	0.84	0.67	0.74	0.82
A_{95}	0.56	0.76	0.59	0.59	0.14	0.77	0.61	0.82	0.66	0.71	0.80

Table 3 - 4: Correlation coefficients for correlation of damage and intensity parameters. Part 1

It can be observed that most values correlate well with each other. This is due-to the fact that the *Spearman rank-order correlation coefficient* was used. Most variables showed a clearly nonlinear dependence on each other as it is shown in figure 3-7 for the local and global mortar damage. A linear correlation coefficient would partially result in significantly lower correlations.

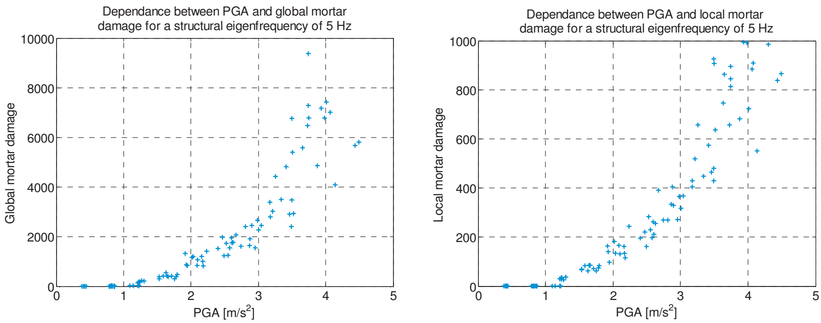


Figure 3 - 7: Correlation of PGA and mortar damage

Also, the correlation over all frequencies is lower than for single frequencies. This effect appears, because the results largely depend on the eigenfrequency of the structure, cf. figure 3-8. Thus results have always to be regarded and compared under the same initial frequency.

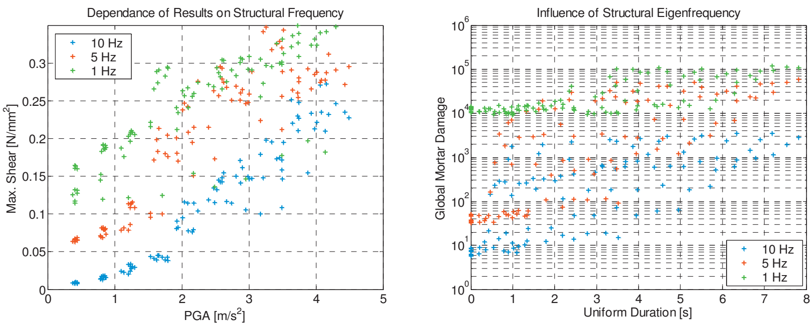


Figure 3 - 8: Dependence of the result on the structural frequency

It is worth noting that the PGA might not have the dominating effect that it is always assumed to have. Instead, the PGV and velocity parameters show a better correlation considering the full band of frequencies. Historical structures, especially churches, have natural frequencies between 1 and 3 Hz and fall in the velocity sensitive region of the response spectra, thus adding importance to these remarks. The fact is further emphasized, if the change of natural frequencies as a result of increasing damage is considered, cf. also figure 4-5.

It is not easy to predict this reduction off the structural frequency. [BOMMER ET AL. 2004a] propose to decrease the frequency of the damaged structure compared to the undamaged structure by a factor of 1.7 to 3.5. For the majority of churches this would mean that they fall within the velocity sensitive region of the response spectrum, where as proved above, velocity is more dominant than PGA.

For very low frequencies, below 0.3 Hz the PGD will have a dominating effect. Now, although the PGD was included in this study, it is difficult to interpret the results. PGD is easily influenced by long term noise and in case of the natural accelerograms it was not always clear which – and if at all – filtering was used and what kind of baseline correction was applied. All artificially generated earthquakes were baseline corrected by a cubic polynomial type and filtered with a fourth order Butterworth bandpass filter between 0.1 Hz and 25 Hz. An even better correlation is obtained, if the RMS values are compared to the damage parameters. This is because some records are dominated by a clear peak of the ground acceleration. RMS values are more powerful to describe the overall distribution of high acceleration values. The same reason explains the also well correlated EDA, A95, SMA and SMV values. *Sustained maximum acceleration (SMA)* and *sustained maximum velocity (SMV)* are defined as the third highest absolute values of acceleration and velocity in the time history. *Effective design acceleration (EDA)* is the maximum acceleration after low-pass filtering with a cut-off frequency of 9 Hz. Finally, *A95* is the acceleration which would include 95% of the Arias Intensity.

Two more general observations may be made. At first, decreasing correlation coefficients for lower frequencies indicate a higher scatter for lower frequencies. This is partially because the structure may react more sensible, but also a consequence of the program code. Local damage was set to a maximum value of 1000 and global damage could not exceed the value of 100 000. As explained in figure 3-6 a damage value of 20 already corresponds to a loss of toughness of 90 percent for a β of 0.8. Thus, there is no need to include values higher than 1000, because the effect on the structure would not alter any more. A sample output for a structural eigenfrequency of 0.5 Hz is given in figure 3-9. As a consequence of the high structural damage which is reached, values of 1000 are often achieved. Since it stops at this value, the correlation is affected and decreases.

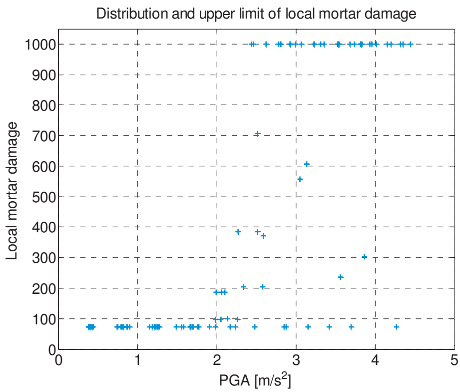


Figure 3 - 9: Effects of upper limit of mortar damage variable

Secondly, global damage shows more dispersion than the local one. As may be seen in the subsequent figure, the local damage is better correlated. This is one of the reasons why local damage is very good indicator for damage as will be further reviewed in chapter 5. Once more, it must be stressed out that the results of this study are valid for the material model used. Based on this study, similar observations may be easily derived for other models as well.

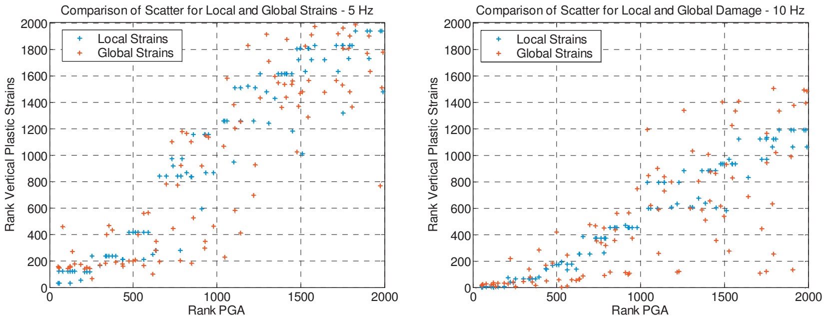


Figure 3 - 10: Difference between local and global damage for varying first natural frequencies

In figure 3-10 the results for the vertical strains are shown. The output would be similar for the internal damage variables and shearing strains. The results show the connection of the rank order correlation coefficients of PGA and vertical plastic strains for two different structural frequencies. The results shown take into account the coefficients, if the full range of 2000 simulations is considered. This will be done for all the following figures on this matter. In contrast to the pictures, tables 3-4 to 3-8 plot the correlation coefficients only for the 200 simulation performed for each structural frequency. The figures have to be compared to the last paragraph of the tables, which is labelled “all frequencies”.

What can be seen in figure 3-10 is that the scatter of the global results is higher. Strains will be included in the further analysis of the results, but they are not as well correlated to the damage parameters and the intensity parameters of the earthquake as it was expected. This is a consequence of the material model. Large strains have to occur only once to cause an increase of the damage variables. Additional strains are only of importance if they further increase the maximum damage obtained before. This is not the case for very small plastic, or even elastic strains. In contrast, the internal damage variables are able to qualify the structural damage well, generally better than the top displacement which is often used as an indicator for structural damage in engineering.

Results achieved so far advise to emphasize the effects of the PGV, since it is best correlated with the structural damage. The velocity was also included in a number of additional intensity parameters, which were introduced in the previous chapters. These paramters were also correlated to the output of the structrural model. The results are listed in table 3-5, again using the rank order correlation coefficient.

	HD	MD	MDS	BD	BDS	CV	CVS	CH	CHS	MS	MSH
10 Hz											
V_m/A_m	0.13	0.10	0.10	0.09	0.09	0.11	0.03	0.16	0.02	0.10	0.11
AI	0.89	0.88	0.88	0.89	0.89	0.90	0.67	0.90	0.77	0.89	0.89
I_D	0.17	0.14	0.14	0.12	0.14	0.15	0.05	0.20	0.11	0.14	0.15
P_D	0.96	0.97	0.97	0.97	0.97	0.98	0.76	0.96	0.86	0.97	0.97
I_C	0.92	0.92	0.92	0.93	0.93	0.94	0.71	0.93	0.80	0.93	0.92
E_D	0.83	0.81	0.81	0.81	0.82	0.83	0.61	0.84	0.71	0.82	0.83
CAV	0.89	0.88	0.88	0.89	0.89	0.90	0.67	0.90	0.77	0.89	0.89
r(t)	0.86	0.84	0.84	0.85	0.85	0.86	0.64	0.87	0.74	0.85	0.86
J_I	0.85	0.84	0.84	0.83	0.84	0.85	0.62	0.87	0.72	0.84	0.85
5 Hz											
V_m/A_m	0.23	0.14	0.20	0.17	0.21	0.18	0.00	0.19	0.05	0.18	0.15
AI	0.94	0.89	0.92	0.92	0.94	0.93	0.72	0.92	0.80	0.93	0.90
I_D	0.27	0.18	0.24	0.21	0.25	0.22	0.00	0.24	0.09	0.22	0.18
P_D	0.97	0.93	0.96	0.97	0.98	0.98	0.85	0.97	0.88	0.97	0.95
I_C	0.96	0.92	0.94	0.95	0.96	0.96	0.76	0.95	0.84	0.96	0.93
E_D	0.89	0.82	0.86	0.86	0.89	0.87	0.63	0.87	0.74	0.87	0.84
CAV	0.94	0.89	0.91	0.92	0.94	0.93	0.72	0.92	0.80	0.93	0.90
r(t)	0.92	0.85	0.89	0.89	0.92	0.90	0.67	0.89	0.77	0.90	0.87
J_I	0.81	0.78	0.83	0.80	0.82	0.78	0.54	0.87	0.69	0.79	0.77
2 Hz											
V_m/A_m	0.40	0.35	0.38	0.33	0.33	0.39	0.12	0.35	0.28	0.36	0.36
AI	0.94	0.87	0.96	0.95	0.97	0.95	0.58	0.88	0.83	0.95	0.93
I_D	0.42	0.37	0.40	0.36	0.37	0.41	0.02	0.39	0.25	0.39	0.40
P_D	0.90	0.80	0.91	0.92	0.95	0.89	0.64	0.84	0.82	0.89	0.89
I_C	0.94	0.86	0.96	0.95	0.97	0.94	0.60	0.87	0.84	0.94	0.93
E_D	0.94	0.87	0.95	0.94	0.95	0.95	0.56	0.88	0.82	0.93	0.92
CAV	0.94	0.87	0.96	0.95	0.97	0.95	0.58	0.88	0.83	0.94	0.93
r(t)	0.94	0.87	0.96	0.95	0.96	0.95	0.57	0.88	0.83	0.94	0.93
J_I	0.90	0.79	0.88	0.87	0.89	0.86	0.64	0.87	0.70	0.86	0.80
1 Hz											
V_m/A_m	0.48	0.32	0.30	0.30	0.23	0.32	0.06	0.40	0.22	0.35	0.28
AI	0.93	0.93	0.81	0.92	0.96	0.93	0.61	0.97	0.82	0.96	0.89
I_D	0.49	0.35	0.34	0.33	0.27	0.35	0.66	0.47	0.24	0.38	0.33
P_D	0.85	0.92	0.81	0.91	0.97	0.92	0.64	0.92	0.85	0.93	0.88
I_C	0.92	0.94	0.82	0.92	0.97	0.94	0.61	0.96	0.84	0.97	0.90
E_D	0.95	0.91	0.81	0.89	0.91	0.91	0.61	0.96	0.80	0.94	0.85
CAV	0.93	0.93	0.81	0.92	0.96	0.93	0.62	0.97	0.82	0.96	0.89
r(t)	0.95	0.92	0.81	0.90	0.94	0.92	0.61	0.97	0.81	0.96	0.87
J_I	0.88	0.73	0.78	0.86	0.89	0.83	0.54	0.85	0.71	0.84	0.64
0.5 Hz											
V_m/A_m	0.58	0.40	0.42	0.38	0.42	0.37	0.12	0.27	0.21	0.32	0.41
AI	0.81	0.86	0.86	0.87	0.52	0.91	0.32	0.92	0.68	0.92	0.86
I_D	0.59	0.40	0.45	0.39	0.38	0.38	0.08	0.31	0.12	0.35	0.45
P_D	0.67	0.79	0.80	0.82	0.45	0.87	0.44	0.88	0.68	0.87	0.78
I_C	0.77	0.86	0.85	0.87	0.51	0.91	0.35	0.92	0.68	0.92	0.86
E_D	0.86	0.87	0.87	0.87	0.58	0.91	0.26	0.89	0.68	0.89	0.85
CAV	0.81	0.86	0.85	0.87	0.52	0.91	0.32	0.92	0.68	0.92	0.87
r(t)	0.84	0.87	0.86	0.87	0.55	0.91	0.28	0.90	0.68	0.91	0.86
J_I	0.88	0.66	0.91	0.80	0.85	0.80	0.44	0.90	0.80	0.78	0.78
All frequencies											
V_m/A_m	0.24	0.19	0.18	0.16	0.03	0.23	0.02	0.24	0.12	0.20	0.22
AI	0.62	0.79	0.61	0.62	0.13	0.84	0.53	0.87	0.65	0.75	0.85
I_D	0.23	0.22	0.19	0.17	0.00	0.26	0.02	0.29	0.13	0.22	0.28
P_D	0.61	0.80	0.62	0.64	0.15	0.85	0.61	0.87	0.69	0.76	0.85
I_C	0.62	0.80	0.63	0.63	0.14	0.85	0.56	0.88	0.67	0.76	0.86
E_D	0.61	0.75	0.59	0.59	0.12	0.80	0.48	0.83	0.62	0.72	0.81
CAV	0.62	0.79	0.61	0.62	0.13	0.84	0.53	0.86	0.65	0.75	0.85
r(t)	0.62	0.77	0.60	0.60	0.13	0.82	0.51	0.85	0.64	0.74	0.83
J_I	0.92	0.78	0.92	0.90	0.93	0.89	0.50	0.84	0.81	0.87	0.89

Table 3 - 5: Correlation coefficients for correlation of damage and intensity parameters. Part 2

As for the parameters analysed before, it can be stated that the local damage mechanisms are correlated better with the intensity parameters than the global ones. Although of all global parameters analysed, those depicted in table 3-6 show the best connection to global, the scatter is still higher than for local damages.

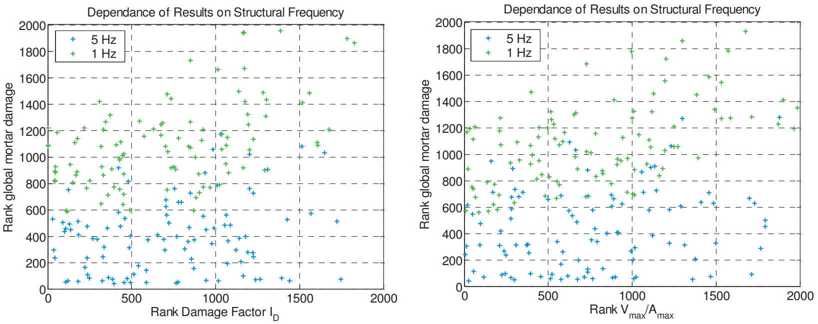


Figure 3 - 11: Rank order correlation for I_D and V_{max}/A_{max}

Results show clear differences between the analysed parameters. The ratio V_{max}/A_{max} is not linked well to any damage indicator. The same is true for the damage factor I_D, although this parameter shows increasing correlation as the frequency decreases. Still, even for low frequencies it is not as well correlated as the other parameters. A sample scatter plot for both parameters is depicted in figure 3-11. Of all parameters the Arias Intensity and energy density exhibit the highest correlation which is underlined in figure 3-12. In general it can be said, that the acceptability of the intensity parameters depends largely on the first eigenfrequency of the structure. Whereas P_D is more appropriate for higher frequencies, the energy density is correlated better for lower frequencies. The measure r(t) shows a good correlation for lower frequencies, while AI, CAV and J_I show a very constant relationship to damages and are thus considered most adequate for the link between intensity and damage. Table 3-6 explains the recommendations for the use of the different parameters. Deep black indicates a good correlation, whereas a light grey corresponds to a lower one.

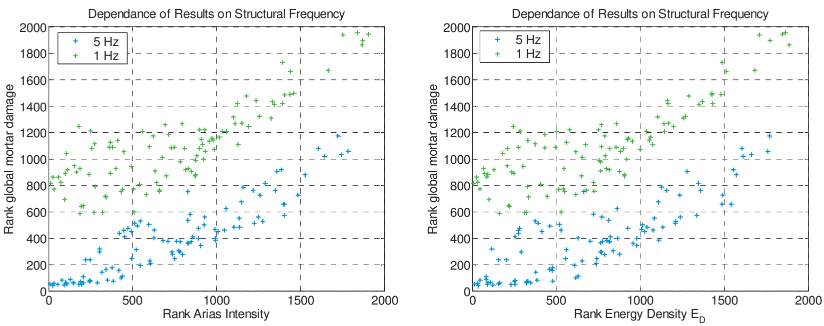


Figure 3 - 12: Rank order correlation for AI and E_D

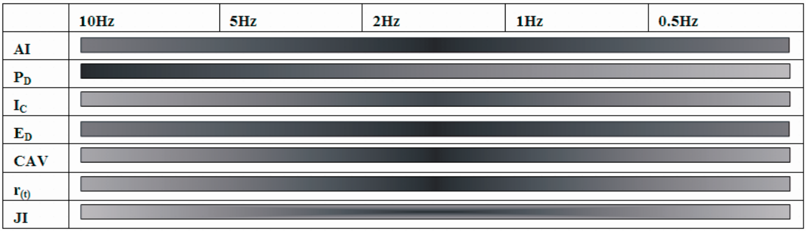


Table 3 - 6: Application recommendation for different time domain intensity parameters

So far, intensity parameters were analysed only for the time domain. Table 3-7 focuses on the frequency domain parameters, which were explained in chapter 3.2.4.

	HD	MD	MDS	BD	BDS	CV	CVS	CH	CHS	MS	MSH
10 Hz											
Sa(f)	0.95	0.95	0.86	0.95	0.97	0.97	0.75	0.93	0.83	0.97	0.95
MP	0.14	0.11	0.11	0.10	0.10	0.12	0.06	0.18	0.08	0.11	0.14
ASI	0.96	0.97	0.97	0.98	0.98	0.98	0.73	0.97	0.86	0.98	0.97
SI	0.91	0.89	0.89	0.90	0.90	0.92	0.69	0.92	0.77	0.90	0.90
5 Hz											
Sa(f)	0.94	0.94	0.91	0.95	0.97	0.96	0.81	0.93	0.86	0.96	0.95
MP	0.26	0.15	0.23	0.19	0.23	0.20	0.00	0.23	0.13	0.19	0.17
ASI	0.97	0.94	0.96	0.97	0.98	0.97	0.85	0.96	0.88	0.98	0.94
SI	0.94	0.90	0.93	0.93	0.95	0.94	0.74	0.94	0.81	0.94	0.91
2 Hz											
Sa(f)	0.92	0.88	0.93	0.94	0.96	0.93	0.62	0.87	0.80	0.94	0.92
MP	0.48	0.43	0.44	0.41	0.39	0.47	0.22	0.43	0.35	0.44	0.45
ASI	0.89	0.80	0.90	0.90	0.94	0.88	0.62	0.84	0.81	0.87	0.89
SI	0.94	0.87	0.97	0.96	0.98	0.95	0.59	0.89	0.84	0.95	0.94
1 Hz											
Sa(f)	0.90	0.90	0.83	0.89	0.91	0.91	0.55	0.95	0.78	0.92	0.86
MP	0.57	0.37	0.34	0.35	0.26	0.38	0.16	0.46	0.28	0.42	0.29
ASI	0.84	0.90	0.81	0.91	0.97	0.91	0.67	0.91	0.85	0.92	0.89
SI	0.93	0.94	0.81	0.93	0.97	0.94	0.62	0.97	0.84	0.97	0.89
0.5 Hz											
Sa(f)	0.78	0.90	0.92	0.92	0.66	0.94	0.32	0.83	0.68	0.89	0.85
MP	0.70	0.46	0.48	0.43	0.46	0.44	0.01	0.32	0.34	0.39	0.41
ASI	0.66	0.77	0.78	0.79	0.42	0.85	0.41	0.87	0.65	0.86	0.77
SI	0.81	0.86	0.86	0.86	0.52	0.91	0.34	0.91	0.70	0.92	0.87
All frequencies											
Sa(f)	0.26	0.64	0.23	0.23	0.31	0.73	0.36	0.59	0.32	0.43	0.77
MP	0.27	0.22	0.20	0.18	0.02	0.27	0.07	0.28	0.17	0.23	0.25
ASI	0.60	0.80	0.63	0.63	0.14	0.84	0.60	0.87	0.69	0.75	0.85
SI	0.62	0.80	0.62	0.63	0.13	0.85	0.54	0.88	0.67	0.76	0.86

Table 3 - 7: Correlation coefficients for correlation of damage and intensity parameters. Part 3

As it may be seen in the table presented above, apart from the mean period of the earthquake all other parameters correlate well with the damage suffered by the structure. Again, it can be seen that the best correlation is found in a measure related to the velocity of an earthquake, the spectral intensity of the velocity. Albeit the frequency parameters produce slightly better correlations than the time domain parameters, this effect is only small, because the correlations to time domain parameters were already high. Again, correlations are not so evident for lower frequencies. Some sample test have shown that the correlation is increased, if the natural frequency of the damaged structure is analysed and the spectral acceleration at the damaged frequency taken. But this is a very tedious task, which does not justify the effort, since good correlations even for low frequencies have been found for the time domain parameters.

The last parameter to be analysed is the impact of the duration of an earthquake. The impact of the duration and its influence on the risk assessment and the evaluation of the structural reliability is still a matter intensively discussed. This task is of special importance for masonry structures, because the influence of stiffness and strength degradation has to be regarded. The assessment of the importance strong motion duration faces some difficulties. As [BOMMER ET AL. 2004a] points out, three problems have to be faced primarily. At first a large variety of different definitions for the time duration exists of which the three – or four respectively – were already introduced in chapter 3.2.1. Since all different definitions are regarded in this study, it does not have an impact at this stage of the work. Secondly, the results will be affected by the material model used to describe the masonry. Energy dissipation through friction, description of the post peak behaviour and degrading strength and stiffness play an important role and models must be capable to reflect these characteristics. The model applied in this research can be seen as one of the most sophisticated damage models which are available, especially in terms of masonry behaviour under random seismic loading. While some improvements could be made, such as 3D effects and the contribution of the mortar head joints, this is a model, which belongs to the upper end of the current state-of art. The last important aspect is that it is hardly possible to decouple duration from other parameters. This is why the approach presented at the beginning of this chapter was chosen. For an earthquake with a total duration of 30 seconds, a uniform duration of 17 seconds, a PGA of 3 m/s² and a response spectrum similar to the black one in figure 3-5, which is then applied to a wall with a natural frequency of 3 Hz the evolution of damage as shown in figure 3-13 is computed.

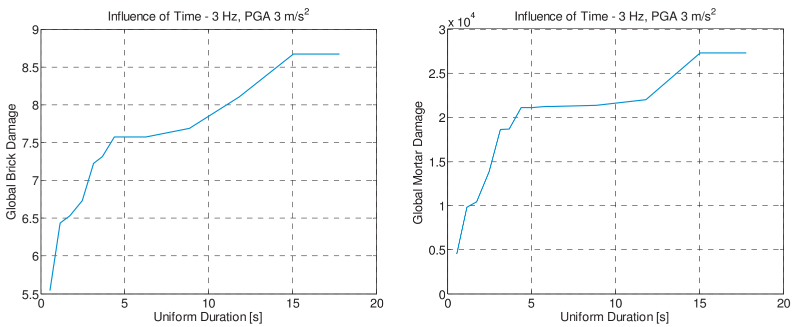


Figure 3 - 13: The influence of duration on brick and mortar damage

	HD	MD	MDS	BD	BDS	CV	CVS	CH	CHS	MS	MSH
10 Hz											
$\mathbf{D_{tot}}$	0.16	0.11	0.11	0.10	0.12	0.12	0.01	0.18	0.09	0.11	0.13
NC	0.15	0.11	0.11	0.09	0.11	0.12	0.01	0.17	0.08	0.10	0.13
$\mathbf{D_{u\ 0.05}}$	0.73	0.69	0.69	0.71	0.72	0.72	0.50	0.73	0.59	0.72	0.71
$\mathbf{D_{u\ 0.10}}$	0.87	0.85	0.85	0.86	0.87	0.88	0.64	0.87	0.72	0.87	0.86
$\mathbf{D_{u\ 0.15}}$	0.93	0.92	0.93	0.93	0.93	0.95	0.72	0.93	0.79	0.94	0.93
$\mathbf{D_{b\ 0.05}}$	0.46	0.41	0.41	0.42	0.45	0.44	0.27	0.46	0.35	0.43	0.44
$\mathbf{D_{b\ 0.10}}$	0.66	0.61	0.61	0.65	0.65	0.65	0.43	0.64	0.49	0.64	0.63
$\mathbf{D_{b\ 0.15}}$	0.80	0.77	0.78	0.79	0.80	0.81	0.58	0.80	0.64	0.80	0.78
$\mathbf{D_{s\ 75}}$	0.13	0.09	0.09	0.08	0.09	0.10	0.00	0.16	0.07	0.09	0.11
$\mathbf{D_{s\ 95}}$	0.16	0.12	0.12	0.10	0.12	0.13	0.03	0.19	0.08	0.12	0.14
5 Hz											
$\mathbf{D_{tot}}$	0.25	0.16	0.22	0.19	0.23	0.20	0.02	0.21	0.06	0.20	0.16
NC	0.25	0.15	0.21	0.19	0.23	0.19	0.02	0.21	0.05	0.19	0.16
$\mathbf{D_{u\ 0.05}}$	0.79	0.76	0.75	0.76	0.80	0.79	0.52	0.77	0.61	0.78	0.77
$\mathbf{D_{u\ 0.10}}$	0.90	0.90	0.88	0.90	0.91	0.92	0.70	0.91	0.77	0.92	0.91
$\mathbf{D_{u\ 0.15}}$	0.94	0.94	0.94	0.96	0.95	0.97	0.79	0.95	0.85	0.96	0.94
$\mathbf{D_{b\ 0.05}}$	0.55	0.48	0.48	0.49	0.56	0.50	0.26	0.50	0.34	0.51	0.50
$\mathbf{D_{b\ 0.10}}$	0.69	0.71	0.65	0.68	0.72	0.72	0.46	0.68	0.53	0.71	0.72
$\mathbf{D_{b\ 0.15}}$	0.81	0.87	0.80	0.83	0.83	0.87	0.64	0.83	0.69	0.86	0.85
$\mathbf{D_{s\ 75}}$	0.26	0.16	0.22	0.20	0.23	0.20	0.02	0.21	0.06	0.20	0.16
$\mathbf{D_{s\ 95}}$	0.26	0.17	0.23	0.20	0.24	0.20	0.02	0.22	0.07	0.21	0.17
2 Hz											
$\mathbf{D_{tot}}$	0.41	0.36	0.38	0.35	0.35	0.40	0.00	0.37	0.26	0.37	0.39
NC	0.39	0.34	0.37	0.33	0.33	0.38	0.01	0.36	0.25	0.35	0.38
$\mathbf{D_{u\ 0.05}}$	0.86	0.87	0.90	0.89	0.88	0.91	0.47	0.84	0.73	0.90	0.85
$\mathbf{D_{u\ 0.10}}$	0.90	0.87	0.95	0.95	0.94	0.94	0.57	0.85	0.80	0.94	0.89
$\mathbf{D_{u\ 0.15}}$	0.88	0.81	0.92	0.91	0.93	0.90	0.60	0.77	0.81	0.89	0.88
$\mathbf{D_{b\ 0.05}}$	0.64	0.70	0.66	0.65	0.64	0.69	0.25	0.65	0.51	0.69	0.64
$\mathbf{D_{b\ 0.10}}$	0.76	0.83	0.83	0.83	0.79	0.84	0.42	0.81	0.64	0.84	0.75
$\mathbf{D_{b\ 0.15}}$	0.81	0.84	0.89	0.88	0.86	0.89	0.54	0.73	0.71	0.87	0.82
$\mathbf{D_{s\ 75}}$	0.40	0.35	0.38	0.34	0.34	0.39	0.02	0.37	0.26	0.36	0.39
$\mathbf{D_{s\ 95}}$	0.42	0.35	0.39	0.35	0.36	0.40	0.00	0.37	0.26	0.37	0.40
1 Hz											
$\mathbf{D_{tot}}$	0.49	0.33	0.33	0.31	0.25	0.33	0.07	0.45	0.22	0.39	0.31
NC	0.47	0.32	0.32	0.30	0.24	0.32	0.07	0.43	0.21	0.31	0.31
$\mathbf{D_{u\ 0.05}}$	0.89	0.81	0.67	0.78	0.85	0.80	0.49	0.89	0.68	0.87	0.78
$\mathbf{D_{u\ 0.10}}$	0.89	0.90	0.74	0.87	0.96	0.88	0.60	0.92	0.78	0.93	0.87
$\mathbf{D_{u\ 0.15}}$	0.84	0.92	0.77	0.90	0.97	0.90	0.66	0.89	0.82	0.93	0.88
$\mathbf{D_{b\ 0.05}}$	0.70	0.56	0.48	0.53	0.58	0.55	0.27	0.72	0.47	0.63	0.56
$\mathbf{D_{b\ 0.10}}$	0.79	0.70	0.55	0.66	0.77	0.81	0.41	0.80	0.58	0.76	0.69
$\mathbf{D_{b\ 0.15}}$	0.84	0.84	0.63	0.78	0.89	0.81	0.52	0.83	0.69	0.86	0.79
$\mathbf{D_{s\ 75}}$	0.49	0.33	0.34	0.31	0.25	0.33	0.07	0.45	0.20	0.37	0.32
$\mathbf{D_{s\ 95}}$	0.47	0.31	0.31	0.29	0.22	0.31	0.06	0.43	0.19	0.35	0.30
0.5 Hz											
$\mathbf{D_{tot}}$	0.59	0.40	0.44	0.39	0.41	0.38	0.13	0.30	0.11	0.35	0.43
NC	0.57	0.39	0.42	0.37	0.39	0.36	0.14	0.28	0.09	0.33	0.43
$\mathbf{D_{u\ 0.05}}$	0.85	0.78	0.75	0.78	0.45	0.80	0.12	0.87	0.62	0.83	0.84
$\mathbf{D_{u\ 0.10}}$	0.76	0.84	0.80	0.83	0.45	0.88	0.28	0.92	0.71	0.91	0.88
$\mathbf{D_{u\ 0.15}}$	0.65	0.83	0.80	0.82	0.45	0.88	0.37	0.87	0.70	0.90	0.84
$\mathbf{D_{b\ 0.05}}$	0.77	0.57	0.55	0.57	0.36	0.57	0.02	0.68	0.35	0.58	0.63
$\mathbf{D_{b\ 0.10}}$	0.77	0.65	0.61	0.65	0.29	0.68	0.04	0.85	0.57	0.74	0.77
$\mathbf{D_{b\ 0.15}}$	0.70	0.77	0.70	0.74	0.34	0.80	0.22	0.85	0.66	0.85	0.85
$\mathbf{D_{s\ 75}}$	0.59	0.40	0.44	0.38	0.41	0.38	0.11	0.29	0.10	0.35	0.43
$\mathbf{D_{s\ 95}}$	0.58	0.41	0.44	0.39	0.42	0.38	0.11	0.29	0.11	0.34	0.43
All frequencies											
$\mathbf{D_{tot}}$	0.23	0.21	0.18	0.16	0.00	0.24	0.03	0.27	0.11	0.20	0.27
NC	0.15	0.17	0.13	0.10	0.04	0.20	0.04	0.24	0.08	0.14	0.23
$\mathbf{D_{u\ 0.05}}$	0.56	0.71	0.53	0.54	0.10	0.74	0.40	0.77	0.55	0.66	0.77
$\mathbf{D_{u\ 0.10}}$	0.60	0.80	0.60	0.61	0.13	0.83	0.53	0.85	0.65	0.75	0.85
$\mathbf{D_{u\ 0.15}}$	0.59	0.80	0.62	0.62	0.14	0.84	0.58	0.85	0.67	0.75	0.85
$\mathbf{D_{b\ 0.05}}$	0.41	0.47	0.34	0.35	0.05	0.50	0.19	0.54	0.33	0.44	0.54
$\mathbf{D_{b\ 0.10}}$	0.52	0.67	0.48	0.50	0.11	0.69	0.37	0.71	0.50	0.62	0.73
$\mathbf{D_{b\ 0.15}}$	0.55	0.75	0.55	0.56	0.12	0.77	0.48	0.78	0.59	0.69	0.80
$\mathbf{D_{s\ 75}}$	0.23	0.20	0.18	0.16	0.01	0.24	0.03	0.27	0.11	0.20	0.26
$\mathbf{D_{s\ 95}}$	0.23	0.21	0.17	0.16	0.01	0.24	0.03	0.27	0.12	0.20	0.26

Table 3 - 8: Correlation coefficients for correlation of damage and intensity parameters. Part 4

Additional information about the influence of duration is given in table 3-8, where the rank order correlation coefficients for the different definitions of time related intensity measures are specified.

Concerning the results, it can be seen easily that the commonly used significant duration has the least correlation of all parameters concerning the damage. Also, the number of cycles and the total duration show only a slight dependence on the data. This could be expected. Otherwise duration would be more important than the amplitude, i.e. long events with a small amplitude would lead to the same damage as a shorter event with higher amplitudes. Now, the bracketed and uniform durations try to reflect this characteristic by taking into account only the time between the exceedance of a certain threshold. For this reason the bracketed duration correlates better and the uniform duration correlates best with the damage parameters. Quite generally, it can also be observed that results correlate better the higher the threshold value is. For practical applications, the threshold should nevertheless be set to 0.05g, else only a smaller number of events may be evaluated and compared.

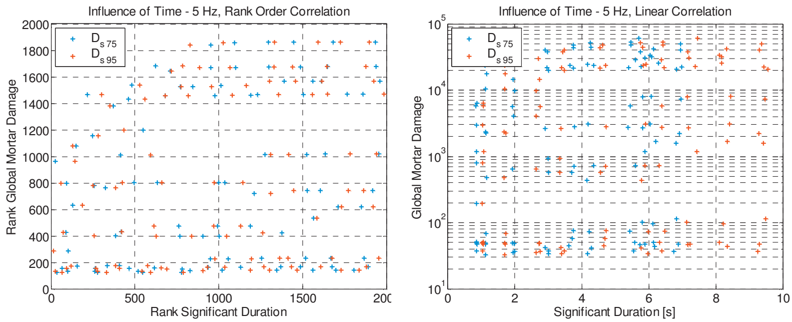


Figure 3 - 14: Rank order and linear correlations for significant duration

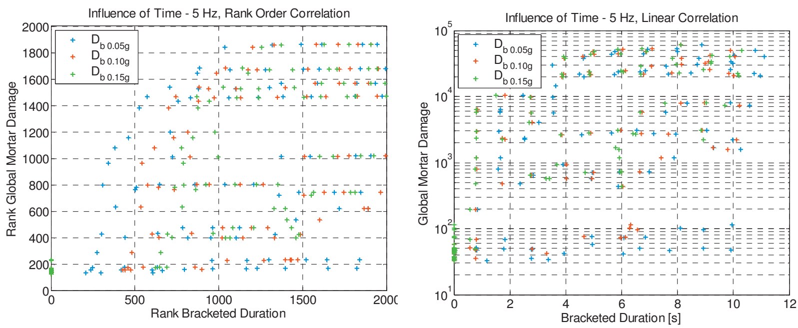


Figure 3 - 15: Rank order and linear correlations for bracketed duration

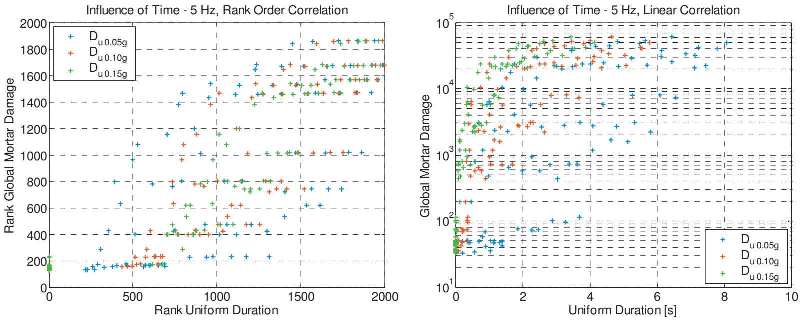


Figure 3 - 16: Rank order and linear correlations for uniform duration

Figures 3-14 to 3-16 show the rank order and linear correlations for all three types of duration with different thresholds with respect to the global mortar damage. What can be seen is not only the increasing correlation with the type of duration but also the decreasing scatter, if the threshold is raised to higher values. Plots of the linear correlation are also shown, because they give an untreated description of the dependence upon each other. The mortar damage for the linear correlation was plotted on a logscale. The same results with a linear scale are presented in figure 3-17.

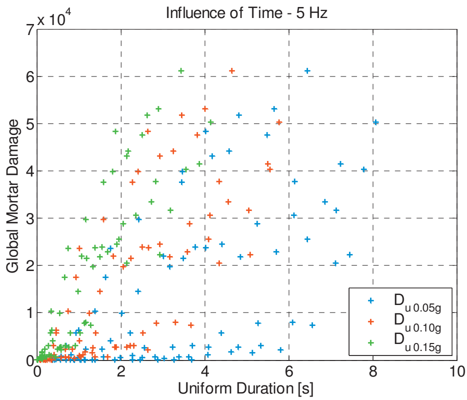


Figure 3 - 17: Linear correlations for uniform duration and mortar damage

The results inform about the connections between duration and global damage parameters, which is close to those of the local effects. The impact of the duration varies with natural frequency and maximum peak ground acceleration, cf. figure 3-18. Interestingly, as shown in figure 3-13 and proven by additional calculations which lead to similar results, the duration has significant influence for shorter duration. Very long earthquakes do not increase the damage much more.

Comparing the results it can be concluded that the acceleration – or velocity as stressed out before – has the primary influence on the damage. Nevertheless, the impact of duration is important and must be regarded in the design of masonry structures. This is even more central for the process of risk assessment.

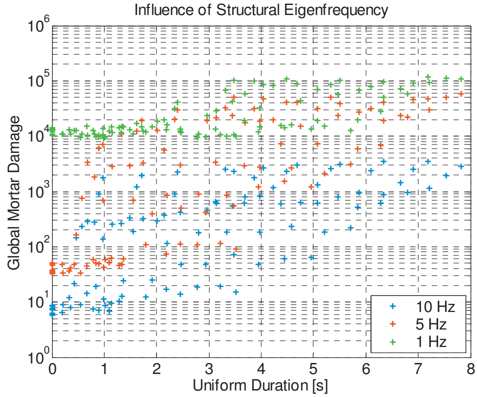


Figure 3 - 18: Influence of duration and natural frequency

If both PGA and duration are observed, the plots shown in figure 3-19 are obtained. The left hand plot leads to the wrong conclusion, that duration has effects only for higher PGA values, thus the picture on the right shows the same plot on a lognormal damage scale. A clear and steady increase of damage with increasing time is visible. The damage described by the two damage parameters inherent in the material model covers both, strength and stiffness reduction.

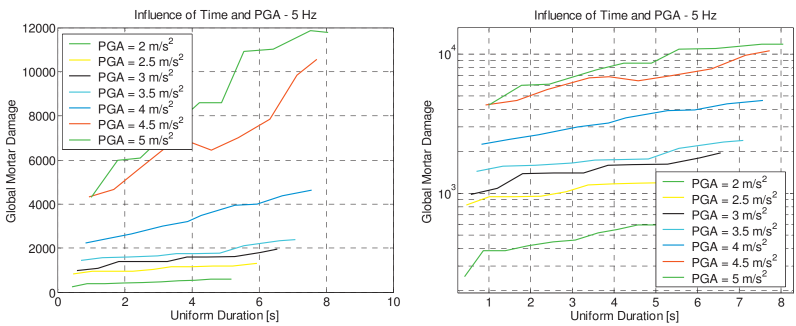


Figure 3 - 19: Influence of duration depending on the PGA

3.4.4 Consequences and conclusions

Several intensity factors were found to be correlated to the damage suffered in the structure. Naturally, the results differ for each considered parameter. Generally speaking, it is concluded that intensity parameters related in some way to the velocity of an earthquake are better correlated than those who take into account the accelerations. Also, the duration was determined to have significant effects on the damage imposed on a structure. Thus, it has to be regarded in the risk assessment of historic masonry structures and parameters for the prediction of the duration must be included. Ultimately, the question whether duration has an effect on the damage has to be answered with yes. This is especially true for the energy based damaged criteria, but naturally less important for the displacement demand. A similar study performed by [BOMMER ET AL. 2004a] draws similar conclusions, although the correlations found were not so significant. This might be because they expressed damage in terms of loss of strength and stiffness. It is also assumed that they used a linear correlation coefficient, but since this information is not given in their text, it cannot be stated for sure.

Combined parameters, whether in the time or in the frequency domain are correlated better to the structural damage than those which refer to a single maximum value, such PGA or PGV. Of all parameters the energy density E_D, the cumulative absolute velocity CAV and the Arias Intensity were the best predictors of structural damage, not only because they showed high correlation coefficients, but also because the coefficients remained nearly constant over the full range of structural frequencies.

Another important parameter is the natural frequency of the structure. It was found to have dominating effects on the size of local and global damage. Also, some intensity parameters are only correlated well within a certain range of frequencies. The scatter of damage for lower frequencies was analysed to be higher. This is in contrast to the results presented by [BOMMER ET AL. 2004a] because their material model focused on shear failure, whereas the model applied here includes also effects due-to tension.

Summarized, it may be stated that most of the existing parameters are well suitable to predict the damage in dependence of the intensity of the ground motion. It was tried to find indices which would provide a better correlation of which the only useful alternative was the product:

$$V = VSI \cdot E_D \tag{3.29}$$

Nevertheless, this product leads only up to a five percent increase in the correlation coefficients for low natural frequencies. Thus the evaluation of the probabilities of occurrence, which is the topic in chapter 3.5, will focus on acceleration, velocity spectrum and the duration of an earthquake.

3.5 Monte Carlo simulation of ground motions

3.5.1 Uncertainty analysis of earthquake ground motion

Generally, uncertainty may be divided into two categories. The *aleatory uncertainty* is due to natural and unpredictable variations in the system studied. It is inherently random and cannot be reduced. Thus, it is often referred to as *aleatory variability* which will also be used in the context of this study. On the other hand the *epistemic uncertainty* is an uncertainty introduced by modelling the system insufficiently. Epistemic uncertainty might also be described as the variability of the outcome of a repeated experiment. It is model dependent and can be eliminated by creating a precise model. In reality this is seldom achieved due to the lack of knowledge of the studied system or the ignorance of facts to achieve good estimates with reasonable effort. Within the decision making process it is important to offer a differentiation between these two types of uncertainties. Decision makers must be informed about the type of uncertainty and its size in order to ease their choice of alternatives. Epistemic uncertainty may also be described by Bayesian statistics, but this is a topic exceeding the scope of this work. The uncertainties which are involved in the assessment of the seismic hazard may be distinguished in three main groups, which are explained in the following figure.

Source uncertainties	Fault type, depth, rupture surface, location, magnitude, occurrence time, occurrence interval, seismic history, geology
Path uncertainties	Distance, regional soil type, attenuation function
Site uncertainties	Local soil profile, topography

Figure 3 - 20: Different types of uncertainties in earthquake engineering

This is also consistent with the source-pathway-receptor model explained at the very beginning of chapter 3. So far the concepts introduced in engineering seismology are reliable in judging probabilities of occurrence around 10^{-5} [BOMMER ET AL. 2004b], [ABRAHAMSON AND SILVA 1997]. Also, if dealt with very low annual exceedance rates, the *probabilistic seismic hazard analysis PSHA* will always involve a large degree of expert judgement [BOMMER ET AL. 2004b].

Concerning the simulation of earthquake ground motions and their probabilities of occurrence, several problems will be highlighted. The three main parameters included in the model will cover the magnitude, distance and soil type, of which magnitude and distance are the most sensitive especially for distances exceeding 20 km [SIGBJÖRNSON ET AL. 2002].

3.5.2 Introduction

In order to assess the probability of certain intensity parameters to occur, a program was developed which is able to express the seismic hazard at the site regarding several input parameters. This program is attached in appendix B. As stated several times before, it is necessary to express risks – for quantitative as well as for qualitative descriptions - in terms of probability distributions. A deterministic observation is a stand alone result which is not capable to explain the complex situation including the diversion of the possible consequences.

Therefore, at first the probability distributions of magnitude and distance have to be determined. Concerning the distance, it has to be differentiated between areas with a uniform seismic hazard and those governed by one or two large faults. Several attenuation function – cf. chapter 3.5.3.2 – for different zones with different seismicity exist. Thus, instead of developing yet another empirical function, the impact of already existing functions was analysed, compared and included in the program; this approach provides also information about the size of epistemic uncertainty by the comparison of different attenuation functions. Alternatively, uncertainty in seismic engineering could be addressed through Bayesian estimation methods as it was for example done by [SIBILIO 2006]. Since Bayesian approaches are not yet so widely known and would require deeper understanding and efforts of statistical methods for the practitioner, they are not pursued here, although in the author's opinion this is a very promising procedure.

3.5.3 Considered input

3.5.3.1 Magnitude

The motivation to include the magnitude distribution can be easily explained by an example of a historical structure located in the lower Rhine embankment in Germany. The seismic hazard is usually expressed in terms of peak ground acceleration relating to a seismic event with a return period of 475 years or a probability of exceedance of 10% in 50 years, which is the background for the response spectra used in the codes. This is due to the fact that the design working life for common structures is set to 50 years in EC 1. In this code it is also stated, that the design working life for monumental structures is given to 100 years. There even are proposals which go as high as 500 years. With respect to a large number of – not necessarily well known – historical churches, this would also seem reasonable. Still, within a period of 100 years, rehabilitation and strengthening are usually performed at least once, so that the 100 year design working life stated in EC 1 is justified.

Applying the Gutenberg-Richter relationship, cf. formula 3.31, with constants a = 2 and b = 0.85 determined by [HINZEN ET AL. 1997], a return period of 475 years would correspond to an event with the magnitude $M_L = 5.5$. If the design working life is now set to 100 years and the probability of exceedance would have to be below 10% in 100 years, this would correspond to a return period of:

$$T_M = \frac{-L_t}{\ln(1-P_f)} = 950\, years \tag{3.30}$$

Where: T_M = return period, L_t = design working life of 100 years, P_f = probability of exceedance in this case 0.1.

The corresponding magnitude would be 5.9. Although the change in magnitude seems to be a minor one, the rise in impact is more obvious if the magnitude is related to the kinetic energy released throughout the earthquake with formula 3.31 or 3.7:

$$\log E = 11.8 + 1.5 \cdot M \tag{3.31}$$

With E = energy measure in ergs; M = magnitude. The ratio of the released energy during both magnitudes is around 4.0. The previous remarks show the importance of including a larger return period for the risk assessment of monumental structures. In buildings codes this is usually done by including an empirical importance factor for monumental structures, which should reflect higher return periods.

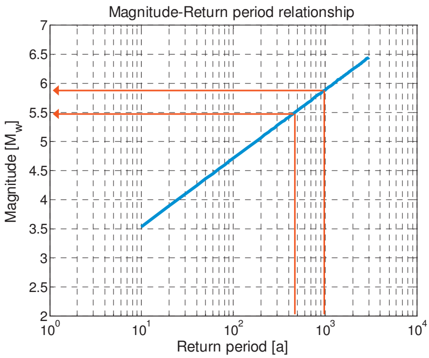


Figure 3 - 21: Influence of the return period on the magnitude

The distribution of the magnitude may be described in different ways. Most common is the *Gutenberg Richter Relationship*:

$$\log_{10}\left(\frac{N}{a_T}\right) = a - b \cdot M \tag{3.32}$$

Where a and b are seismic constants determined for each region by a combination of historical earthquake data and modern instrumental records, N is the number of events in time period a_T, which is given in years and M is the Magnitude, normally assumed to be the Local Magnitude. Alternatively, formula 3.32 may be expressed as:

$$\left(\frac{N}{a_T}\right) = 10^{a-b\cdot m} = \exp(\alpha - \beta \cdot m) \tag{3.33}$$

The Gutenberg-Richter relationship is an exponential distribution, thus the probability density distribution between a lower and an upper magnitude may be determined including the upper magnitude m_u an the lowest considered magnitude m_0 as:

$$f_M(m) = \frac{2.303 \cdot b \cdot \exp(-b \cdot \ln(10) \cdot (m - m_0))}{1 - \exp(-b \cdot \ln(10) \cdot (m_u - m_0))} \tag{3.34}$$

The probability density functions based on the Gutenberg Richter relationship should be truncated as it is done in the previous formula, otherwise magnitudes above physically possible events will be predicted. The mean annual rate of exceedance can be expressed in terms of upper and lower magnitude as:

$$\lambda_M = \nu \frac{\exp(-b \cdot \ln(10) \cdot (m - m_0)) - \exp(-b \cdot \ln(10) \cdot (m_{\max} - m_0))}{1 - \exp(-b \cdot \ln(10) \cdot (m_u - m_0))} \tag{3.35}$$

with $\nu = \exp(\alpha - \beta * m_0)$

In some cases another type of distribution is preferred. [SÁNCHEZ-SILVA AND RACKWITZ 2004] use the *extreme value distribution type III* for maxima, which is sometimes referred to as *inverse Weibull* distribution. In this case, the conditional probability density function of the magnitude is given as:

$$f_{M(m|m>M_{\min})} = \frac{\frac{w}{M_u - u} \cdot \left(\frac{M_u - m}{M_u - u}\right)^{w-1} \cdot \exp\left(-\left(\frac{M_u - m}{M_u - u}\right)^{w}\right)}{1 - \exp\left(-\left(\frac{M_u - M_{\min}}{M_u - u}\right)^{w}\right)} \tag{3.36}$$

Where M_u = upper magnitude, M_{min} = lower magnitude. The constants w and u describe the shape of the graph and are thus determined by the maximum likelihood method to best fit the observed data of seismicity in the given region. The value used for M_{min} varies in different applications. It ranges from a non specified Magnitude of M = 4 in [SÁNCHEZ-SILVA AND RACKWITZ 2004], $M_S = 4.5$ [SUCKALE ET AL. 2005], to M = 5 in [KARIMI 2005]. For this study, a minimum magnitude of $M_S = 4.5$ for the uniform spatial distribution of the seismic hazard was used. In case of a governing fault, the minimum magnitude was set to $M_S = 5.0$, to avoid a large number of very low ground motions.

Comparing both approaches, it can be concluded that the Gutenberg-Richter relationship is widely applicable, whereas the assessment with a Weibull distribution seems more appropriate for larger areas with a significant seismic hazard.

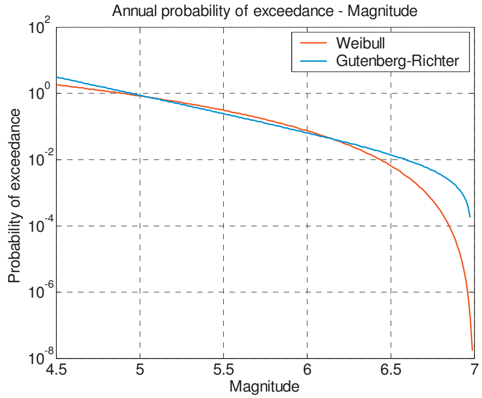


Figure 3 - 22: Comparison of Weibull and Gutenberg-Richter equation

Figure 3-22 was based on the analysis of a region with strong seismic hazard using the following Gutenberg Richter constants: a = 5.45, b = 1.1. The Weibull parameters were determined as w = 4.5 u = 1. Weibull predicts slightly higher probabilities for lower magnitudes and shows a steep line for higher magnitudes, it is thus more accurate in estimating very low probabilities, which are not explicitly dealt with here. Resulting yearly probabilities of exceedance for an area of 75 km radius with a uniform seismic hazard using several different attenuation functions for both magnitude distributions are shown in figure 3-23. It can be seen that effects occur only for very low probabilities below 10^{-5} which are not of major interest for this study.

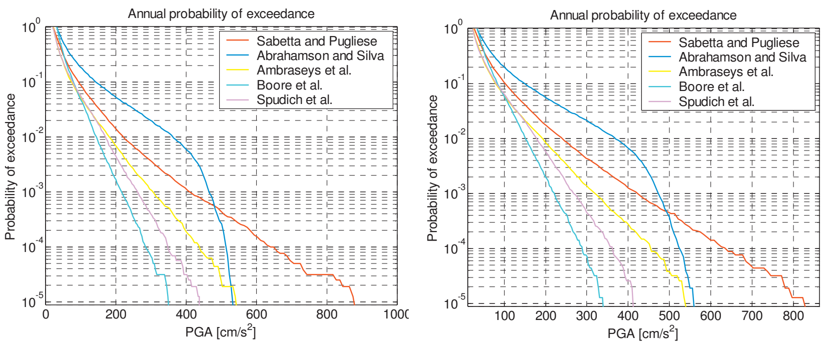


Figure 3 - 23: Effects of magnitude distributions on PGA occurrence on stiff soil, Gutenberg-Richter distribution of magnitude (left), Weibull distribution of magnitude (right)

Within the seismic hazard assessment one of the most crucial factors is the determination of upper bounds of earthquake ground motion. It is thus an important task to find the upper bounds for possible ground motions and earthquake properties. The best overview of these factors is given by [BOMMER ET AL. 2004b]. The factors limiting extreme ground motions are on the one hand the maximum seismic radiation that might occur at the fault, i.e. the magnitude, style of faulting, depth of faulting, distance and site conditions. On the other hand interactions of waves and travel paths will also limit extreme ground motions. Finally, these upper limits are determined by the maximum possible motion that can be transmitted to the shallow geological layers. Interestingly, resulting limits to the PGA are quite low as listed in the following table according to [BOMMER ET AL. 2004b].

Study	Year	Soil Type	PGA [g]
Ambraseys	1970	Very soft marine deposits	0.15
		Inorganic clays of low and medium plasticity	0.30
		Deposits of high plasticity	0.50
Ambraseys	1974	Normally consolidated clays	0.15
		High plasticity clays	0.30
		Saturated sandy clays and medium dense sands	0.50
Mohammadioun and Pecker	1984	Near source alluvial site	0.50
Dowrick	1987	High plasticity normally consolidated clays	0.36
		Medium dense sands and saturated sandy clays	0.61
		Overconsolidated clays	1.89

Table 3 - 9: Limits for PGA [BOMMER ET AL. 2004b]

It can be seen, that based on the increasing amount of available recordings the limiting values for ground motions have increased with time. The highest recorded PGA was 1.8 g [BOMMER ET AL. 2004b].

The determination of an upper bound is most important for deterministic seismic hazard assessment, where usually the worst case scenario is tried to be identified and the structural integrity measured at this event. As stated by [BOMMER ET AL. 2004b] the upper bounds for a probabilistic seismic risk assessment play a major role only for frequencies of exceedance around the order of 10^{-7} and 10^{-8}, which is done for example for nuclear waste depository projects. Within this study the only upper bound implemented is the bound on the maximum possible magnitude which should be determined by historical earthquake data and measurements of a given size.

3.5.3.2 Attenuation functions

The decrease of wave energy as a result of the distance travelled through a certain medium is described by attenuation functions. In this research five different attenuation functions for the PGA were applied, which shall be explained in the following. Although they are commonly used for the determination of possible response spectra, they may also be applied to predict the PGA or other types of intensity measures. Attenuation functions are based on the summation of several of the components of the general form:

$$\ln Y = C_1 + C_2 \cdot M + C_3 \cdot M^{C_4} + C_5 \cdot \ln\sqrt{R^2 + h^2} + C_6 \cdot \sqrt{R^2 + h^2} + f_{(Source)} + f_{(Soil)} + \varepsilon \tag{3.37}$$

Where:

Y	=	Intensity parameter
C_1	=	To adjust the unit of Y
C_2-C_4	=	Exponential relation between magnitude and energy
C_5	=	Geometrical dispersion in distance R and depth h
C_6	=	Decrease of energy in dependence of distance R and depth h
f	=	Functions describing source mechanisms and soil type
ε	=	Error

In detail, the following attenuation functions were used. At first, the algorithm developed by [Sabetta and Pugliese 1996] was implemented in the program:

$$\log_{10}(Y) = a + b \cdot M + c \cdot \log_{10}\sqrt{R^2 + h^2} + e_1 \cdot S_1 + e_2 \cdot S_2 \pm \sigma \tag{3.38}$$

With M being the local magnitude, R the epicentral distance in kilometres, σ the standard deviation of the logarithm of Y and S1 and S2 referring to the site classification. The algorithm may be used for determination of PGA, PGV, AI and duration. Depending on the desirable outcome the values of table 3-10 for the parameters a, b, c, e_1 and e_2 must be used line-by-line. The data shown are plotted only for larger horizontal component of the earthquake ground motion and the epicentral distance; the same values are available in [Sabetta and Pugliese 1996] for the vertical components and the fault distance.

Parameter	**Constant Term a**	**Mag. Coeff. b**	**Dist. Coeff. c**	**Site Coeff. e_1**	**Site Coeff. e_2**	**H**	**Sigma**
PGA	-1.845	0.363	-1	0.195	0	5.0	0.190
PGV	-0.828	0.489	-1	0.116	0.116	3.9	0.249
AI	0.729	0.911	-1.818	0.244	0.139	5.3	0.397
Duration	-0.783	0.193	0.208	-0.133	0.138	5.1	0.247

Table 3 - 10: Constants for equation 3.37

This algorithm was developed for European earthquakes and magnitudes between 4.6 and 6.8 and distances below 100 km. This does not mean that beyond these borders the algorithm is not correct, but the database the algorithm was created on includes only magnitudes and distances within these borders. The magnitude is equal to M_S when both M_S and M_L are greater or equal to 5.5 and M_L in all other cases.

The next function was developed by [ABRAHAMSON AND SILVA 1997] for distances below 100 km and a moment magnitude M_w between 4 and 8. The general form is given by:

$$\ln(Y) = f_{1(m,r_{rup})} + F \cdot f_3(M) + HW \cdot f_{4(m,r_{rup})} + S \cdot f_{5(pga_{rock})} \tag{3.39}$$

This formula takes also into account the closest distance to the rupture plane in km r_{rup}, the fault type F and hanging wall effects HW. Finally, soil type S is also included. The constants itself are dependent on a number of additional formulas. The interested reader is referred to the program code attached in appendix B or the source cited above. Also developed for European earthquakes is the algorithm by [AMBRASEYS ET AL. 1996] valid for range of magnitude M_S from 4.0 to 7.5 and source distances r up to 200 km. S_a and S_s are constants defining the soil type.

$$\log_{10}(Y) = -1.48 + 0.266 \cdot M_s - 0.922 \cdot \log(r) + 0.117 \cdot S_a + 0.124 \cdot S_s + 0.25 \cdot P \tag{3.40}$$

Finally, two more approaches are considered. [BOORE ET AL. 1997] developed an approach for moment magnitudes M_w between 5.5 and 7.5 for distances no greater than 80 kilometres which is characterized by the following function:

$$\ln(Y) = b_1 + 0.527 \cdot (M - 6) - 0.778 \cdot \ln r - 0.371 \ln(V_s / 1396) \tag{3.41}$$

The factor b_1 corresponds to the mechanisms leading to the earthquake, because it is not further considered in this study a value of -0.242 is taken. V_s is the shear wave velocity in the medium. The last function was developed by [SPUDICH ET AL. 1999] for moment magnitudes M_w greater than 5 and distances less than 100 km.

$$\log_{10}(Y) = 0.237 + 0.229 \cdot (M - 6) - 1.052 \cdot \log_{10} \sqrt{R^2 + 5.57^2} + 0.174 \cdot \Gamma \tag{3.42}$$

Where Γ is a factor equal to 0 for rock sites and equal to 1 for soils.

Next to the algorithm of Sabetta, two other algorithms were applied to assess the Arias Intensity attenuation. The first is presented by [ZONNO AND MONTALDO 2002] and is very similar to the one of [SABETTA AND PUGLIESE 1996]:

$$\log_{10}(Y) = 0.713 + 0.664 - 1.046 \cdot \log_{10} R + 0.075 \cdot \Gamma \tag{3.43}$$

In their work [TRAVASAROU ET AL. 2003] state that aleatory variability in the Arias intensity is larger than for most other ground motion parameters, especially compared to the PGA and S_a. The formula implemented is:

$$\begin{aligned} \ln(Y) = {} & 2.8 - 1.981 \cdot (M - 6) + 20.73 \cdot \ln(M / 6) - 1.703 \cdot \ln \sqrt{R^2 + 8.78^2} \\ & + (0.454 + 0.101 \cdot (M - 6)) \cdot S_c + (0.479 + 0.334 \cdot (M - 6)) \cdot S_D \\ & - 0.166_1 \cdot F_N + 0.512 \cdot F_R \end{aligned} \tag{3.44}$$

Where M denotes the moment magnitude, S_C and S_D are indicator variables for the soil type and F_N and F_R are variables indicating the fault type.

Since the duration was evaluated to have significant effect on the damage of a structure, three functions were also implemented – beside the Sabetta approach – to predict the strong motion duration at the given site. One of these is an algorithm based on the seismic moment M_0 developed for earthquakes in Iceland by [ÓLAFSSON ET AL. 2001]:

$$\log_{10} T_D = -4.727 + 0.0517 \cdot \log_{10} M_0 + 0.663 \cdot R \tag{3.45}$$

Another proposal based on European earthquakes was offered by [HERNANDEZ ET AL. 2001].

$$\ln(Y) = -1.04 + 0.44 \cdot M + 0.19 \cdot \ln D + 0.04 \cdot S \tag{3.46}$$

With M being the local magnitude for events less than value of 6 and the surface save magnitude in all other cases. D is the epicentral distance and S a component considering the soil. Finally, the last attenuation function for time is considered to be the one by [MIDORIKAWA AND KOBAYASHI 1979]:

$$d = 0.013 \cdot 10^{0.42 \cdot M} + 0.24 \cdot D \tag{3.47}$$

With M being an unspecified magnitude and D the distance from subfault to observation site.

3.5.3.3 Distance

Two models for the spatial distribution are employed. The first is governed by a uniform spatial distribution of earthquake events. In that case the probability density function of the distance is simply determined by the term:

$$f_r(R) = \frac{2 \cdot r}{R_{\max}^2} \tag{3.48}$$

The second one is in case of a single fault. In this case, the probability density function of the distance is given by formula 3.49.

$$f_r(R) = \frac{2 \cdot r}{L \cdot \sqrt{r^2 - r_{\min}^2}} \tag{3.49}$$

3.5.4 Results

Some general remarks on the importance of the parameters will be given, to emphasize the importance of the correct input and to provide information about the variation of the results. Effects of the scatter in magnitude were already explained. The distance is another parameter of greatest importance for the outcome of the simulations. The decrease of AI and PGA with increasing distance is shown in figure 3-24.

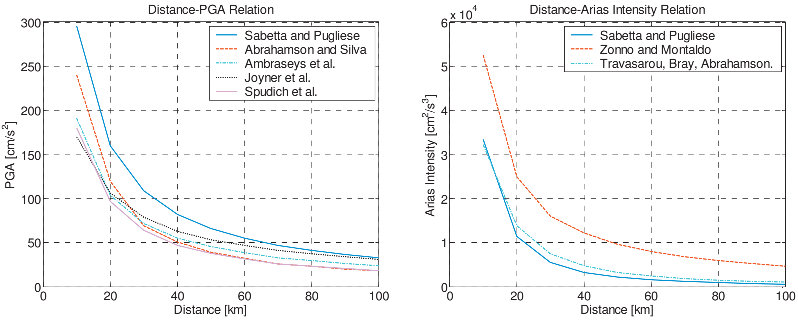


Figure 3 - 24: Influence of distance on PGA and AI for stiff soil

The effects of the distance also differ, if the hazard is dominated by a single fault in a close distance to the site. Using formula 3.49, with a minimum distance of 10 km and a total length of the fault of 150 km, the results shown in figure 3-25 are obtained. Their main characteristic is the flat slope at the beginning, indicating a second peak at higher ground accelerations if several events are calculated. The small blue histogram shown in the figure represents the outcome of 10^6 simulations.

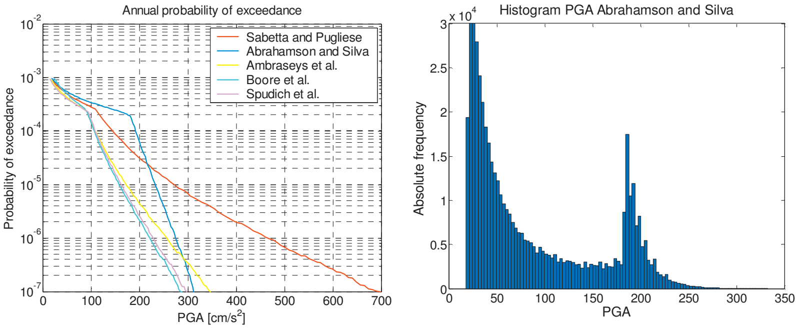


Figure 3 - 25: Probability of exceedance of PGA for single fault and stiff soil

Because of the high number of variables included, it is hardly possible to give general results on the outcome of the calculations. The only information valuable for a larger number of simulations is that the distribution of distance has to be assigned a key role in the hazard assessment. Also, epistemic uncertainties are rather high, since the algorithms may differ significantly in their results, as figures 3-25 to 3-27 show, representing PGA, duration and Arias Intensity predictions.

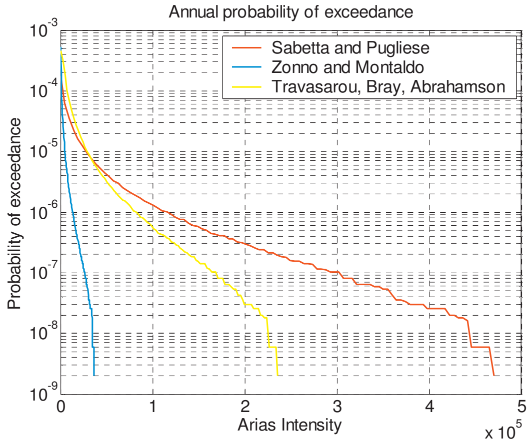


Figure 3 - 26: Predictions of Arias Intensity for stiff soils

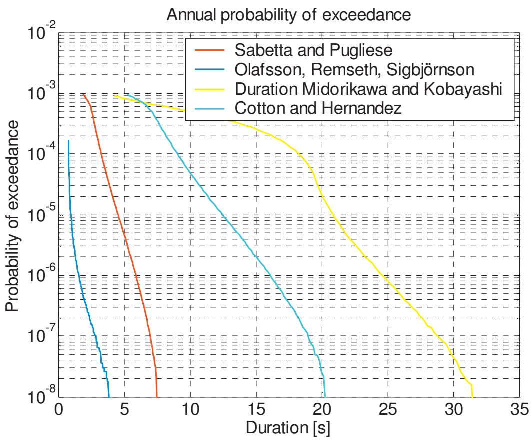


Figure 3 - 27: Prediction of duration for stiff soils

The epistemic uncertainty is of special importance for very low probabilities. It is thus recommended to regard probabilities of hazard occurrence below 10^{-5} with greatest care. Also, the predictions of the ground motion duration vary largely. This is basically a result of different durations for higher probabilities. The slope is similar for most of them, if lower probabilities are considered. The duration used in the applied algorithms refers in most cases to the significant duration $D_{s\ 95}$. Some additional results are depicted in appendix E. Those results include the effects of soil, fault type and several fault distances.

3.5.5 Conclusions

The calculations revealed the high dependency of the results on the applied attenuation function and the inherent epistemic uncertainty. It is crucial to offer decision makers information about this type of uncertainty. This information may at least be provided qualitatively, if this approach is used. Also, it is now possible to predict the probability of occurrence of several intensity parameters describing the variability of the earthquakes itself which is a great improvement especially regarding the effects of strong motion duration which have been proved to have a significant effect on the damage suffered by the structure.

The results may now be applied as load parameters in the next chapter, where the evaluation of the structural vulnerability and the impact of the variability of material parameters are described.

Chapter 4

Vulnerability assessment

4.1 Introduction

4.1.1 General considerations

The evaluation of the structural vulnerability is certainly the core task of engineers within the risk assessment process which was described in chapter 2. The vulnerability was defined as the sensitivity to a given load event – in this case earthquakes – expressed by damage depending on intensity. Not surprisingly, research in engineering focuses often on this matter [Augusti et al. 2001], [Lourenco and Oliviera 2005], [Lagomarsino et al. 2002], [Lang 2002], [Sadegh-Azar 2002] to name only few.

The approach used and the results obtained depend largely on the calculation method and the material model included. Four methods are available which are presented in figure 4-1.

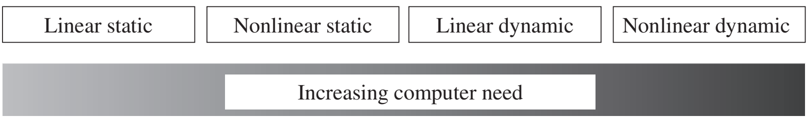


Figure 4 - 1: Computation methods

Interestingly, the evaluation of the structural safety of historical buildings relies often on linear static methods. In Mediterranean countries nonlinear static procedures are now widely applied, although they nearly always depend on plasticity theory. [Kuhlmann et al. 2003] used a linear dynamic approach to analyse the behaviour of the Aachen Cathedral in Germany with respect to ground motions. A larger study of nonlinear dynamic performance was never carried out. The main reason is that the computation time remains high and the scatter of results is expected to be so high that nonlinear dynamic evaluations would not provide significantly better information than nonlinear static ones. The task is especially difficult for historical buildings, as for example Augusti [Augusti et al. 2001] compromises in five major points, which in the following are depicted with slight adjustments:

1. Each monumental, e.g. truly unique and not only historical, structure is characterized by its own history. This leads to large differences in geometry, material, due to added or changed structural elements and interacting parts of the building.

2. Interpreting the static and especially the dynamic behaviour is too complicated to be expressed by simple mechanical models. Consequently, it is not only inadequate to apply modern building codes to historical buildings, nor is it generally possible to extrapolate results of procedures developed for modern buildings.
3. The material, structural and geometrical properties are not known and although experimental data may lead to improvements this will neither be complete nor sufficient to describe the structure in all details.
4. Especially for masonry structures, the structural resistance decreases with time and use as a consequence of deterioration.
5. Because of the peculiarities and characteristics of each structure, it is rarely feasible to extrapolate reliable damage data from past earthquakes.

The starting point of this work is the lack of knowledge of the structural behaviour under random loads and uncertain material properties. Fundamental results of the structural response of typical elements depending on changing load and material parameters will be obtained. Therefore, it will be compulsory to use nonlinear dynamic procedures in order to include all major effects in a first step and determine those, which are most important for further evaluations. It is not stated that the nonlinear dynamic analysis is the sole solution for the assessment of the seismic risk. In contrast, simple methods for practical application with a low possibility of introducing human errors into the system have to be developed. Nevertheless, it must be underlined with respect to the current state of art that the seismic analyses of historical constructions remains a great challenge. Thus, in a first step advanced modelling is necessary to receive information about the complex behaviour and possible damage of historic structures with respect to seismic excitations. The models will be used as a numerical laboratory to perform virtual experiments. Sensitivities, scatter of parameters and the structural behaviour have to be analysed to gain valuable information about the process as a whole.

In this context, it must be stressed out again that risk assessment in general and this thesis in special do not deal with the proof of seismic safety, but with the prediction of damages, which requires a more detailed evaluation of the structural response.

4.1.2 Considered material models

A critical factor in the determination of the structural vulnerability is the applied material model and its capabilities and drawbacks in the description of the material. Large efforts have been and are still made in studying the material behaviour of masonry structures. Several material models have been developed, of which some are quite similar or only applicable to a special kind of structure, while others offer sophisticated models which are not freely distributed for public use. For this study, a model is needed, which is able to predict the post-peak behaviour and the degrading of strength and stiffness with respect to cyclic loading. Also, it should describe the masonry as a smeared material, since large structures will be modelled and a differentiation between bricks and mortar requires too much modelling and computation time. The smeared orthotropic material should be able to sufficiently describe the mortar-stone interaction. Several models were tested and the following short discussion shall explain the reasons, which lead to the final choice of the material model developed by [GAMBAROTTA AND LAGOMARSINO 1997], which was developed further by [CALDERINI AND LAGOMARSINO 2004].

At first, linear elastic models were applied. Although – without doubt – they are only capable to give a rough approximation of the structural behaviour in a static and even more in a dynamic point of view, they provide some advantages. They enable engineers to gain a rather simple and fast insight into structural parts which are, due to concentration of peak stresses, damaged easily. Additionally, a linear modal analysis of a full church offers important information about structural frequencies as for example done by [KUHLMANN ET AL. 2004], [KUHLMANN ET AL. 2003].

Elastic brittle models are often applied for concrete structures. They are for instance applied by [BARTHEL 1993] and based on the 5 parameter yield surface of [WILLIAM AND WARNKE 1974]. These models lead easily to numerical instabilities, if the calculations focus on vaults and arches [JAGFELD 2000]. Thus, an elastic-plastic material is often employed, which admits different stress-strain relationships for different stress states. This approach is for example applied together with the finite element program ANSYS® by [BERGANDER 1995] for the Frauenkirche in Dresden, Germany. Finally, damage models exist, which describe the dependency of stresses and strains on given damage variables, which in turn depend on the inner material state, such as cracks. A major advantage of damage models is their ability to provide additional information about the damage occurring in the structure.

For larger structural models, elastic-brittle, elastic-plastic and damage models may be implemented into smeared crack models. The name is assigned because cracks are described by integrating the strains over the catchment area of the integration points of an element. Normally, they are used for models which are dominated by a larger number of smaller cracks. Computing might face serious difficulties, if less – but larger – cracks occur. Smaller elements may result in higher strains at one integration point because the damage, i.e. crack, is more localized. This might lead to numerical instabilities. Additionally, the size of the crack is not described very well any more.

Sometimes discrete elements are used and proposed, e.g. [SCHERMER 2004]. Instead of modelling a continuum, masonry is idealized by blocks. In this case, the blocks are idealized by rigid elements and the nonlinearity is introduced by the contact between the blocks. Compressive stresses and friction are allowed, but commonly no tension. This modelling is applicable for smaller structures. Due-to the setup of the numerical model and the resulting computation time this approach is rather tedious for larger models. Numerous other models were developed and applied to all kind of static and dynamic problems. A good overview about static deployment is given by [SCHLEGEL 2004]. Additional detailed insight into masonry material models is provided for example by [SCHERMER 2004] and [LOURENCO 1996].

Regarding the abilities of the model in comparison to elastic-plastic, elastic-brittle and linear elastic models, and after a detailed study showed that the results are not dependent on the mesh size [CALDERINI AND LAGOMARSINO 2004], it was chosen to use the material model developed by [GAMBAROTTA AND LAGOMARSINO 1997]. Chapter 4.1.4 will introduce the material model applied more thoroughly, after some additional remarks on computational strategies have been given in the following chapter.

4.1.3 Computational strategies

The choice of the correct material model does not only depend on whether a static or dynamic analysis is performed. Furthermore, the scale of the model and the type of information that has to be obtained are core requirements to be considered. According to EC 8, three different levels for the assessment of the vulnerability are allowed [JCSS 2003]. Since it is rarely possible due to the missing understanding of complex structures to perform a *level I* observation, which would only include data of structural size and material, more detailed investigations have to be performed. They could include measurements and in situ tests, *level II*, or even a detailed modelling of the structure reflecting the nonlinear behaviour, which would correspond to a *level III* investigation. It was already explained that this research focuses on detailed models of the structure and material. Nevertheless, the need for simplified modelling is obvious, if large structural models want to be evaluated. The differentiation between micro-, meso- and macromodelling of masonry structures is assumed to be well known, it is for example explained in [SCHUEREMANS 2001] or other sources mentioned in chapter 4.1.2. Without doubt, a micro modelling approach for larger masonry structures is not useful in the assessment of church structures, since this would include distinguishing between bricks and mortar and their interface. The result would only be an unjustifiable amount of man-hours in creating the structural model and – at the present time – computation time. The latter applies also for meso modelling. Consequently, only macro modelling based on a homogenization procedure of the stratified masonry medium is possible. All these considerations emphasize why the model of [GAMBAROTTA AND LAGOMARSINO 1997] was chosen. The model is rather easily implemented into the finite element program ANSYS®, has proven its robustness in sample tests and was verified through several of numerical and experimental results.

4.1.4 Applied material model

Although this work deals only with an application and not the development of a material model, the background shall be explained briefly, so that parameters, advantages and drawbacks will be well known. The model was previously mentioned and cited. It aims at modelling large scale masonry structures which are assembled through vertical walls and vaults or domes. Static as well as dynamic analyses are feasible. Different masonry patterns are taken into account by the input of tested material parameters. Otherwise, the model is a continuum model based on an equivalent stratified medium. Plane stress is assumed, but to account for out-of-plane behaviour the masonry may be modelled with multilayered shell elements. The model is able to describe strength and stiffness degrading, cf. figure 4-2, tensile versus compressive behaviour of masonry, as well as the hysteric response to cyclic shearing strains. Hysteric dissipation is possible through activated frictional mechanisms. Evolution of damage is described in terms of two damage variables denoting the brick damage and the damage in the mortar joints. In this case only the contribution of the bed joints is included. A recent improvement of the model including the head joint contribution will be available soon. For modelling, it is necessary to know the orientation of the mortar bed joints throughout the structure.

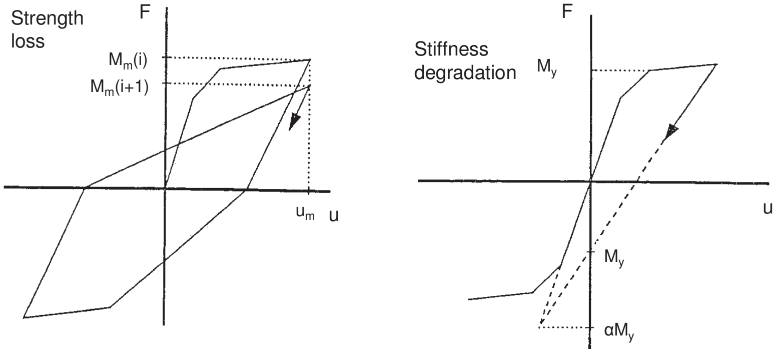


Figure 4 - 2: Strength loss and stiffness degradation

The strains, cf. formula 4.1, occurring in the element throughout the numerical solution are divided into an elastic contribution and an inelastic contribution, which is described by three internal variables which evolve through the damage process and are also iteratively fitted in each load step.

$$\varepsilon = C_M \cdot \sigma + \varepsilon^*{}_m + \varepsilon^*{}_b \tag{4.1}$$

Where C_M is the elastic compliance matrix, ε^*_b describes the inelastic brick and ε^*_m the inelastic mortar contribution which may be further divided into extensions ε and sliding γ.

$$\varepsilon^* = \{0 \quad \varepsilon \quad \gamma\} \tag{4.2}$$

They are determined for the mortar contribution by formulas 4.3 and 4.4. Similar equations may be derived for the brick and the brick damage and not explained further.

$$\varepsilon_m = c_m \cdot \alpha_m \cdot H(\sigma_2) \cdot \sigma_2 \tag{4.3}$$

$$\gamma_m = c_m \cdot \alpha_m \cdot (\tau - f) \tag{4.4}$$

In these formulas H is the Heaviside function to reflect the unilateral response of the interface, c_m is the inelastic compliance parameter for extensional and tangential mechanisms in the mortar brick joint. f represents the friction at the interface, it vanishes if tensile stresses occur and limits the sliding in case of compressive stresses. α_m is the mortar damage variable, which has been discussed in chapter 3 already.

The effect of sliding of the brick is negligible compared to the mortar bed contribution, thus if ε_m, ε_b and γ_m are known, the strains in the element may be determined. The inelastic contributions are governed by the evolution of the damage variables α_m and α_b throughout the load process. They are connected to the damage energy release rate and the toughness function $R_m(\alpha_m)$, see also figure 4-3. If the energy release rate is less or equal than the toughness function, damage takes place. The dependence was explained in figure 3-6 and formula 3.28. The dissipated energy and the damage energy release Y_m rate have to be determined within the infinitesimal load step and the damage variables iteratively calculated.

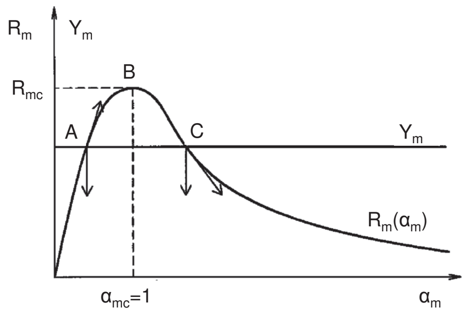


Figure 4 - 3: Damage function for the mortar joint, stable (A) and unstable (B) evolution

Generally speaking, if tensile stresses act on the mortar bed joints, then both damage mechanisms of brick and mortar become active. If mortar bed joints are subjected to compressive or tensile vertical and horizontal loads, than three different damage mechanism may become active: the sliding of the bed joint, the damage to the bed joints and damage of the bricks. The failure domains are represented in figure 4-4.

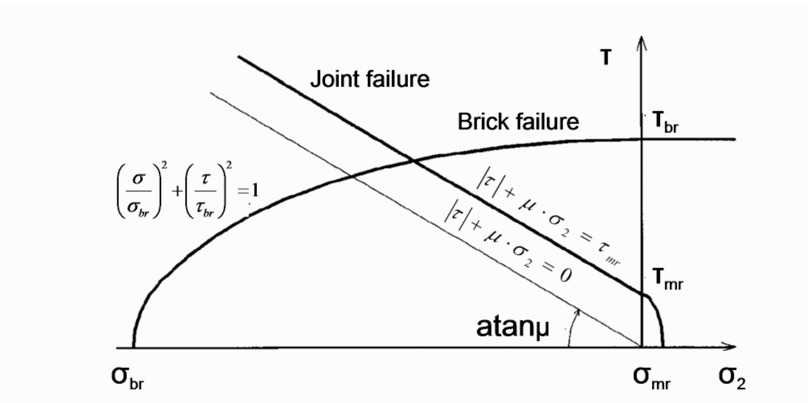


Figure 4 - 4: Mortar joint and brick failure domains

To successfully apply the model, the material parameters written in table 4-1 should be known. This table also presents typical values for the parameters used frequently in this study.

Parameter	Description	Typical value for historical masonry
E	Young's modulus of masonry	2000 N/mm²
μ	Friction coefficient	0.6
η	Poisson ratio	0.1
ρ	Density	2000 kg/m³
τ_{br}	Shear strength of bricks	1.5 N/mm²
τ_{mr}	Shear strength of the mortar joints	0.20 N/mm²
σ_{br}	Compressive strength of masonry	3.5 N/mm²
c_{bn}	Failure strain masonry	1.0
β_b	Softening coefficient of the masonry.	0.4
σ_{bm}	Tensile strength mortar	0.15 N/mm²
c_{mt}	Failure shear strain mortar	1.0
β_m	Softening coefficient mortar	0.8

Table 4 - 1: Material parameters used

4.1.5 Comparison of pushover and time history techniques

The pushover analysis as the most important member of nonlinear static procedures has become a popular tool for the assessment of existing structures and the design of new buildings. The technique is especially powerful for *performance based seismic engineering (PBSE)*. With respect to this thesis, it is important to point out the drawbacks and abilities of this calculation method particularly in comparison to nonlinear dynamic procedures.

Expressed in a very simplified manner, it is fair to say that a pushover analysis applies a horizontal load to structure and increases it until the structure fails. Now, the force-displacement relationship obtained is transferred into the *spectral acceleration versus spectral displacement* - also called *Acceleration Displacement Response Spectra*, or shortly, *ADRS* – and the structural performance may be assessed. The approach is explained in numerous publications of which the most basic are [FAFJAR ET AL. 1990], [WEN 2005] and [FEMA 2000]. In contrast to the dynamic analysis, it requires only monotonic constitutive models. The models need not to be capable of representing the unloading-reloading behaviour of structures. The result is, of course, that the static procedure requires simpler models and less computation time, which is one of the main reasons for the success of this procedure.

Still, some drawbacks have to be pointed out. Of course, static analyses always neglect dynamic effects. Thus, mass distribution is only reflected roughly by assuming a load pattern correlating the first eigenmode of the structure. Moreover, neither damping effects are included; nor is the duration of the strong motion reflected. Finally, changes in the modal properties of the structure as a consequence of increasing damage will not be taken into account. In this way, collapse mode and elastic eigenmode of the structure are somewhat confused. The following picture shows a *short time Fast Fourier transformation* of the top displacement of the triumphal arch shown in figure 4-6 during an earthquake.

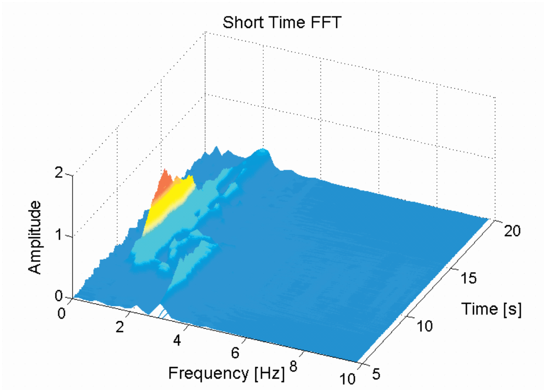


Figure 4 - 5: Changes in the natural frequency of a structure during an earthquake

It can be seen that the structure starts with an initial natural frequency of approximately 3 Hz. After damage is suffered, this frequency drops to 1 Hz and shows also spectral amplification. Both effects are neglected within a pushover analysis. Additionally, it has to be stated that the assumed load pattern in a pushover analysis might differ from the real one, if the structure is loaded dynamically. This is above all the case for higher structures with frequencies around 1 Hz and important contributions of higher modes to the structural behaviour. As the last point, pushover analysis is not able to reflect torsional effects for more irregular structures, although newer approaches try to overcome this drawback.

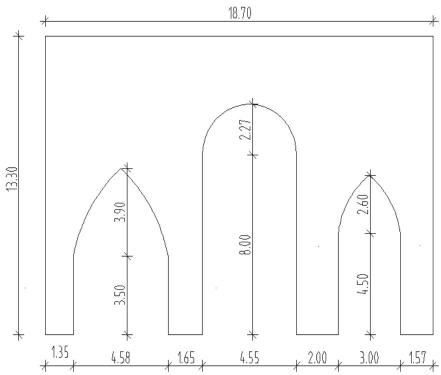


Figure 4 - 6: Geometry of the triumphal arch analyzed after information of [GIORDANO ET AL. 2001] all values in units of metres

To compare the results of dynamic and static calculations the base-shear versus top displacement plot of the dynamic method has to be transferred to the ADRS used in the pushover analysis. Spectral acceleration and spectral displacement are coupled as shown in formula 3-19. For the capacity spectrum conversion, each point is conversed with respect to the first mode spectral ordinates. The necessary formulas are:

$$S_{ai} = \frac{V_i}{W} \cdot \frac{1}{\alpha_i} \tag{4.5}$$

$$S_{di} = \frac{\Delta_{roof}}{(PF_1 \cdot \phi_{1,roof})} \tag{4.6}$$

With V_i denoting the base shear, W the weight of the building, α_i and PF_1 the modal mass coefficient and the participation factor for the first or i-th natural mode of the structure, Δ_{roof} the top displacement at the roof level and $\Phi_{1,roof}$ the roof level amplitude of the first mode. A similar conversion can be performed for the records of the time histories. The results of a time history and a pushover calculation may be compared in this way. This was done for the triumphal arch in figure 4-6. The applied earthquake time-history reflected an earthquake with a surface magnitude of 6.0 and a distance to the epicentre of 20 km on stiff soil. The differences between the two techniques and results are depicted in figures 4-7 to 4-9.

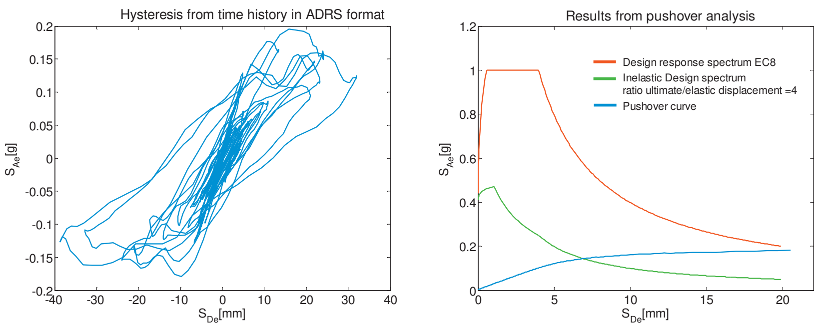


Figure 4 - 7: Comparison of time history results (left) and pushover analysis (right) in ADRS format

Figure 4-7 shows results for both techniques in the ADRS format. The pushover results are already related to a design response spectrum. The inelastic spectrum was derived by taking the ratio of the ultimate to elastic displacement as four. This information can also be gathered from the pushover curve in the right plot of figure 4-7. To compare the results better, both data will be plotted together in figure 4-8. The diagram needs to be explained, because the plot is to some degree confusing. The design response spectrum is still shown in red, while the inelastic spectrum remains green. Plotted in magenta is the pushover curve, which can now be compared to the hysteresis of the time history in blue and the response spectra of the earthquake in black. The shape of the spectra of the real earthquake may look odd because it is plotted in the ADRS format.

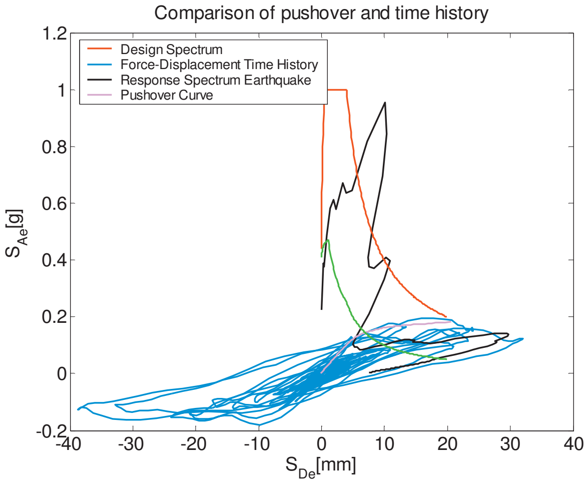


Figure 4 - 8: Comparison of time history and pushover I

For a more direct comparison of the data figure 4-9 may serve. This plot is essentially the same plot as figure 4-8, without the response spectra of the design and natural earthquake and with the data plotted only in the range of the pushover curve.

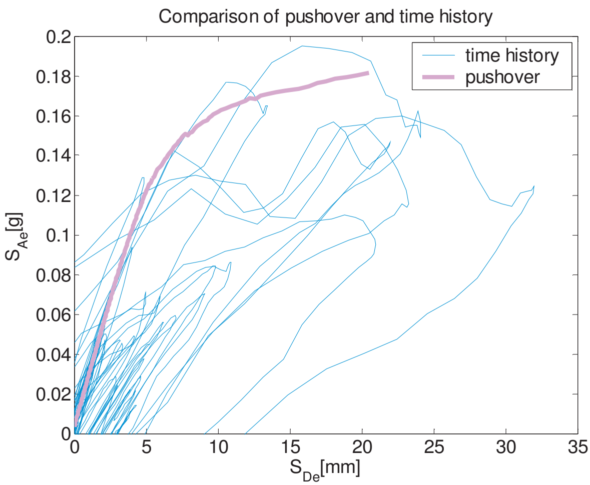


Figure 4 - 9: Comparison of pushover and time history II

The results indicate the differences between the two models. The stiffness degradation of masonry results in greater displacements while transmitting less base shear, which is expressed in figures 4-8 and 4-9 by the spectral acceleration. In this way, the nonlinear dynamic procedure is capable of expressing the damage more accurately. The performance point for the pushover curve is close to the elastic displacement, while in reality the structure suffers severe damage. Now, it has to be admitted that the results of the pushover analysis cannot be compared directly to a single earthquake, which by chance might lead to higher excitations than predicted by the response spectrum. Still, the presented results clearly show the drawbacks for the prediction of damage to masonry structures, if the pushover method is used.

4.1.6 Number of simulations

The task now is to perform a sufficient number of simulations to ensure a reliable output of data. The great amount of simulations planned and the long computation time for each numerical experiment required some consideration on which type of method could be used to decrease the total number of simulations. Due-to the large variety of structural elements and the diverse properties describing the material behaviour, it was clear from the beginning that it is impossible to perform the number of simulations required to assess the overall failure probability. Instead, the goals are to calculate the mean and standard deviation of the output parameters correctly and to provide a sufficient estimate of upper and lower values in order to replace them by probability density functions which might then be used to calculate the failure probability.

The three most common techniques for the reduction of variance and decreasing the size of simulation are *Latin Hypercube Sampling*, *Importance Sampling* and *Adaptive Sampling*. In the *Direct Monte Carlo* simulation all sampling points are placed at random locations which might lie close to each other. This method is sometimes also referred to as *Crude Monte Carlo Method.* This is due-to the fact that the sampling has no memory; it creates each sample without relating it to previous one. The Latin Hypercube sampling overcomes this drawback by subdividing the range of all random input intervals into n intervals with equal probability, thus it is also called stratified sampling. For every variable the interval is now only hit once. Adaptive Sampling tries to find most useful locations for the sampling points throughout the study. Future sample locations are thus estimated at each step iteratively. The Importance Sampling technique is similar, but it assumes a priori distribution of the samples, while in Adaptive Sampling the new samples are made up on the fly. Moreover, importance sampling focuses on those factors, which have the highest influence on the mean value of the outcome.

The technique chosen was the Latin Hypercube sampling, although adaptive or importance sampling are the more powerful tools. It was a great convenience that the method is already implemented in the finite element program ANSYS®, which is also necessary for the application of the masonry material model. Tests revealed that the method is well suited, as figure 4-10 shows. It can be seen that with 250 simulations, the mean and the standard deviation are evaluated nearly as precise as it is done after 1000 simulations. The same is true not only for minimum and maximum values of the shear stresses, but also for all other parameters evaluated. The minimum number of simulations to be performed for each model was set to 1000, in order to obtain reliable information and to include possible outliers, e.g. due-to a more complex structure.

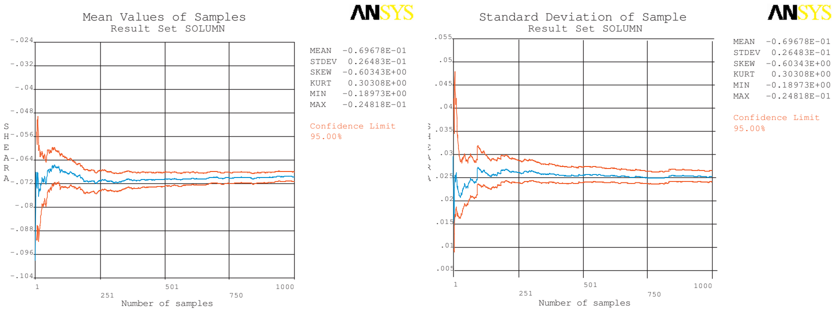


Figure 4 - 10: Development of mean value and standard deviation for the shear stresses over the range of 1000 simulations

The advantages of nonlinear dynamic analyses and their importance for the evaluation of expected damages have been stressed out several times throughout the last chapters. Another question arises now, which is about how to deal with the computation time for larger structures. An innovative approach was proposed by Italian investigators, to divide a structure into major elements, which are then analysed. This *macroelement* approach will be presented in the following.

4.2 Identification of macroelements

4.2.1 Overview of churches

Every single church is unique. To compare their structural layout is hardly possible. Instead churches differ in a great variety of matters. Nevertheless, the structural and seismic behaviour of churches is governed significantly by elements which are recurring frequently. These elements are called macroelements after [DOGLIONI ET AL. 1994], who presented this approach for the first time. A simple layout is presented in the subsequent figure.

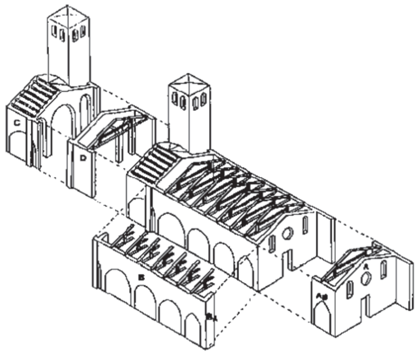


Figure 4 - 11: The principle of macroelements after [DOGLIONI ET AL. 1994]

Still, the simple layout presented in the previous figure cannot be applied in all cases. While the upper example is related mostly to a common layout found in Italian churches, the ground plan of German churches is more as shown in figure 4-12.

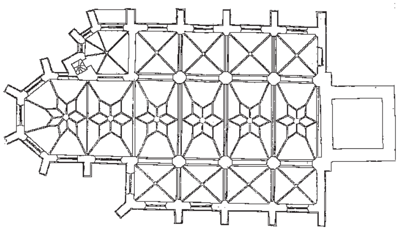
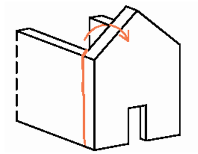
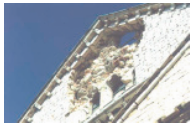
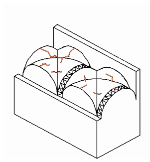
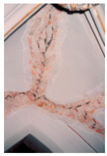
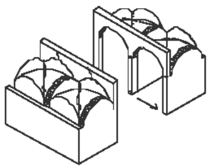


Figure 4 - 12: Typical floor plan of a German church, here St. Martinus in Linnich, Germany

This is the most frequent type of German church as it was found out by a study presented in chapter 6. Another often recurring type in Italy consists of a single dome at the intersection of main longitudinal nave and transversal nave. Fortunately, different geometrical layouts can be considered with the macroelement approach. If these elements are categorized, they will be referred to as *typological macroelements*. In some cases, it might be difficult to determine those elements, which are governing the structural behaviour mostly due to an extraordinary layout or very complicated geometry. If this occurs, simple numerical simulations have to identify the elements based on their behaviour. This type of element is referred to as *behavioural macroelements*. Both will be shortly presented in the following.

4.2.2 Typological macroelements

[LAGOMARSINO ET AL. 2002] propose the following eight *typological macroelements* which result in a total of 18 *damage mechanisms*. Pictures of all mechanisms may be found in this source.

Macroelement	Number	Damage mechanisms	Example
Facade	1	Overturning of the facade: The whole facade (typical part of a church, e.g. for most Italian constructions, front or entrance, built separately from the rest) is separating from the lateral walls due to a lack of connection or anchoring.	
	2	Overturning of the gable: due to rose windows or lack of connection with the roof, the upper part of the facade is overturning or partially destroyed.	
	3	Shear mechanisms: cracks in the facade in the typical X shape	
Nave/Transept	4	Transversal vibration of nave or transept: if lateral walls are too slender or transversal tie rods are missing. Indicators are cracks parallel to the walls of the nave and close to the base of the pillars.	 
	5	Longitudinal vibration of the central nave: indicators are cracks in the longitudinal arches opened cracks at the base of the column and diagonal shear cracks in the vaults of the lateral waves.	
	6	Vaults of the central nave: disjointedness from stiffening arches, if vaults are too low or to thin. Sometimes due to concentrating actions from the roof covering.	 
	7	Vaults of the lateral naves; the same as described above.	

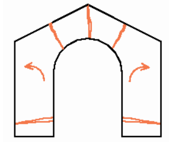
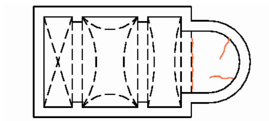
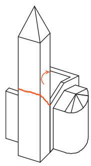
Triumphal arch	8	Kinematism in the triumphal arch: formation of hinges since the arch is next to the nave the main factor for the stiffness of the church in transversal direction.	
Dome/Tambour	9	For a major collapse a continuous arched crack has to occur. Otherwise cracks in the tambour.	
Apse	10	Overturning of the apse.	
	11	Damage to the vaults of the apse.	
Widespread	12	Overturning of other walls: chapel's walls.	
	13	Shear failure of the side walls.	
	14	Damage in the roof covering.	
	15	Interaction of damage with different structural behaviour.	
Bell Tower	16	Global collapse of the bell tower, especially if the tower is separate from the rest of the building.	 
	17	Mechanisms in the bell cell, localized failure of the tower in the upper region, where the bells are located.	
Projection	18	Overturning of standing out elements, e.g. finials.	

Table 4 - 2: Typological macroelements [LAGOMARSINO ET AL. 2002]

4.2.3 Behavioural classification

Sometimes it might not be possible to separate the building into governing parts. Or it is suspected that the structural parts of the building interact in such a way, which would forbid a simple determination of typological elements. In that case, a behavioural classification has to be performed. For a first step, a simple modal analysis or linear dynamic test will reveal the highest stressed parts. It is accepted that this is only a very rough simplification not reflecting the nonlinear structural behaviour. Nevertheless, it is still commonly applied and despite the drawback well suited to determine elements that will be analysed in a more accurate modelling.

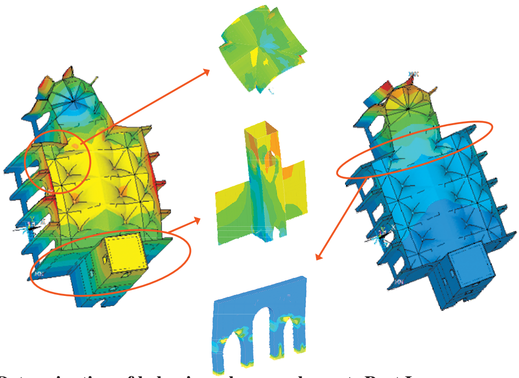


Figure 4 - 13: Determination of behavioural macroelements Part I

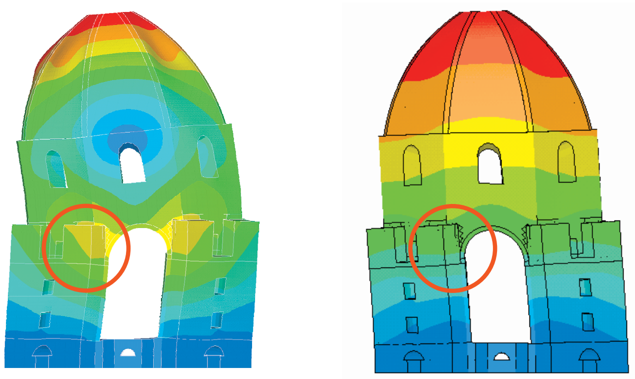


Figure 4 - 14: Determination of behavioural macroelements Part II

Both figures above show an example for the determination of behavioural macroelements. The upper one plots the total displacement of the first (right – frequency 1.9 Hz) and second (left – frequency 2.6 Hz) eigenfrequencies of the church, whose ground plan was laid out in figure 4-11. Corresponding to table 4-2 it can be easily seen that the abside and the tower are the elements excited most. This statement was confirmed by the fact that exactly these two parts of the building were damaged in the Roermond earthquake 1992, which had a local magnitude of 5.9 and an epicentre located at a distance of 40 km to the structure [GD NRW 1993]. Additionally, the church was damaged in an even smaller event, the 2002 Alsdorf earthquake with a surface magnitude of 4.9 and an epicentral distance of approximately 30 km. Another element of major importance is the arch linking the abside and the main nave. Directly adjacent to this arch the vault is found, which is most strongly excited by ground motion. Although the vaults found in the middle of the nave exhibit a larger displacement, the occurring stresses are less, for the differential displacement at the pillars or walls supporting them are not as high as those of the corner vault. The same is true for the vault between abside and nave.

Figure 4-14 shows a very different layout of a church structure. Represented is the Cappelle Medicee in Florence, Italy, showing the total displacements of the first (right – frequency 1.0 Hz) and second (left – frequency 2.6 Hz) eigenfrequency. Due to the complex displacement of the dome and its effects on the tambour, the parts of highest importance for the structural safety are the cupola itself and those parts of the structure with an opening towards the side domes. Generally, domes and vaults should be analysed closer, because they exhibit a higher vulnerability. The natural frequencies of the structures explain another prominent reason for the high susceptibility towards damage of these structures. The main energy contents of earthquakes fall within the same range as the natural frequencies of the structure resulting in strong excitations of the buildings.

4.3 Scatter of input parameters

4.3.1 First remarks

A number of considerations that lead to the final layout of the working plan for the vulnerability assessment have been explained. The effects on the structure are now evaluated. Of utmost importance was to understand the influence of material parameters on the numerical result and its probable consequences for the outcome of the risk analysis procedure. [ROTS 1997] quotes a sceptic view of new approaches in the numerical modelling of masonry as follows:

> "Given the large scatter of properties of masonry, an accurate numerical approach is senseless."

Indeed material test prove every time that the properties of historical masonry scatter extremely. This is especially because of the great variety of materials used for stone and mortar, but also owing to geometrical aspects, e.g. brick pattern and ratio of brick height to mortar layer height.

Within this study, the impact of several parameters is assessed. It can be distinguished between the intensity parameters determining the load factors, which were analysed in chapter 3, the geometrical parameters determined by the structure itself, structural parameters, mainly damping and finally the parameters describing the material behaviour. The parameters used are listed in table 4-3. The calculations include only the variations of PGA and PGV, because at the time these calculations were carried out, the results of chapter 3 were not yet fully known.

Type	Symbol	Variable
Load	X_{skal}	Horizontal PGA
Load	Y_{skal}	Vertical PGA
Load	T	Duration $D_{s\ 95}$ of the ground motion record
Load	E_D	Specific energy density
Load	PGV	Peak ground velocity
Material	μ	Friction coefficient
Material	σ_{mr}	Tensile strength of mortar joints
Material	τ_{mr}	Shear strength of mortar joints
Material	σ_{br}	Compressive strength of masonry
Material	τ_{br}	Shear strength of masonry
Material	ν	Poisson ratio
Material	ρ	Density
Material	E_x	Young's Modulus
Material	E_y	Young's Modulus
Material	β_m	Softening coefficient mortar
Material	β_b	Softening coefficient brick
Material	c_{mt}	Inelastic distortion
Material	c_{bt}	Inelastic distortion brick
Structural	adamp	Rayleigh mass damping
Structural	bdamp	Rayleigh stiffness damping
Geometry	t	Thickness of the layers (mainly for vaults)

Table 4 - 3: Definition of parameters, whose impact is assessed within this study

Naturally, these parameters differ in all important statistical terms, such as mean value, standard deviation, minimum and maximum values or even the type of distribution for each building. The same is true for the diverse kinds of materials which are used for common masonry from sandstone to volcanic brick or marble. It is necessary to gather some information about the possible range and type of scatter of the above mentioned properties. For this reason, a larger literature study was performed to gain valuable information about the material properties, which will be presented in the following.

4.3.2 Literature review

Although the scatter of material parameters can be very high, a sight of literature reveals that even for historic masonry some generalizations concerning the distribution of parameters may be made. [PROSKE 2002] presents a good overview of distributions proposed for various types of concrete and masonry material properties which are depicted in table 4-4.

The normal distribution und lognormal distributions are dominant, but not the only ones offered. Different distributions are also found in [GUAN AND MELCHERS 1997], but they are proposed only for very special solutions based on a smaller number of tests. The normal distribution is applied for parameters whose influence depends on a sum of random effects, where no effect has a dominating influence. It is especially useful for the assessment of errors in measurements and often adopted as a convenient approximation for distributions, which cannot be determined well. In some cases the validity of the normal distribution might not be sufficient. This is for example important, if a distribution demonstrates a significant tail into one direction. In this case, the lognormal distribution is applicable. It is often used for strength of plastic materials, yield stresses and flood peak discharges [BENJAMIN AND CORNELL 1970] to name only few examples.

Material Property	Probability distribution	Source
Concrete compressive strength	Normal	[Stewart 1995], [Spaethe 1992] [Onken and Rostasy 1994] [Noakowski 1988] [Plate 1993], [Lu et al. 1994] [Rüsch et al. 1969] [Barlett and MacGregor 1995]
Concrete compressive strength	Lognormal	[Fischer 1995], [Fischer 1999] [Rackwitz 1998], [Östlund 1991] [Kandarpa et al. 1996] [Bergmeister 1997] [Viesmann and Zilch 1995] [Crespo-Minguillon and Casas 1998]
Concrete tensile strength	Normal	[Onken and Rostasy 1994] [Noakowski 1988]
Concrete tensile strength	Lognormal	[Rüsch et al. 1969], [Kiefer 1997]
Masonry strength (general)	Normal	[Kirtschig 1991] [Franke and Goretzky 1992]
Masonry strength (general)	Lognormal	[Tschötschel 1989] [Franke et al. 1991]
Concrete elastic modulus	Normal	[Merzenich 1995], [Östlund 1991] [Guan and Melchers 1997]
Concrete elastic modulus	Lognormal	[Kandarpa et al. 1996] [Kiefer 1997]
Compressive strength masonry stone	Normal	[Grunert 1982]
Compressive strength masonry stone	Normal/Lognormal	[Möller et al. 1998]
Density	Normal	[Bergmeister 1997] [Viesmann and Zilch 1995] [Schneider 1996]
Dimensional deviation	Normal	[Bergmeister 1997] [Viesmann and Zilch 1995] [Maaß and Rackwitz 1980]

Table 4 - 4: Proposals for probability distribution. Modified and added after [PROSKE 2002]

The sources cited above show the most popular distributions for some of the material parameters considered. Still, more information on the statistical moments or minimum and maximum values has to be included. Therefore, more literature was sighted. The results are shown in table 4-5.

Parameter	Distribution	Mean	Dev.	Min.	Max.	Source
σ_{mr}	lognormal	2.6	0.49			[SCHUEREMANS 2001]*
σ_{mr}		3.9	0.58			[SCHUEREMANS 2001]*
σ_{mr}	normal/lognorm.	0.28	0.10			[SCHUEREMANS 2001]*
σ_{mr}	lognormal			0.10	0.20	[TRINGALI ET AL. 2000]
σ_{mr}				0.08	0.14	[TOMAZEVIC 1995]
σ_{mr}		0.30	0.07			[ROTS 1997]
σ_{mr}		0.22	0.10			[ROTS 1997]
τ_{mr}	lognormal			0.15	0.88	[ROTS 1997]
μ				0.74	1.00	[ROTS 1997]
σ_{br}	lognormal	6.34	2.37	~2.0	~17.0	[SCHUEREMANS 2001]*
σ_{br}	pareto	6.34	2.37	~2.0	~17.0	[SCHUEREMANS 2001]*
σ_{br}	lognormal	5.16	1.56			[SCHUEREMANS 2001]*
σ_{br}	lognormal	8.00	1.93			[SCHUEREMANS 2001]*
σ_{br}	lognormal	4.26	0.83			[SCHUEREMANS 2001]
σ_{br}		4.54	0.77			[SCHUEREMANS 2001]
σ_{br}		4.54	0.77			[SCHUEREMANS 2001]
σ_{br}		12.1	2.2			[BINDA 2000]
σ_{br}		5.2	0.6			[GREIFENHAGEN 2001]
σ_{br}		5.1	3.1			[GREIFENHAGEN 2001]
σ_{br}	lognormal			10	20	[TRINGALI ET AL. 2000]
τ_{br}		0.50	0.15			[SCHUEREMANS 2001]
ρ	normal	1720	28.9			[BINDA 2000]
ρ		1968	21.1			[GREIFENHAGEN 2001]
ρ		1573	32.7			[GREIFENHAGEN 2001]
ρ		1858	23.5			[GREIFENHAGEN 2001]
ρ		2270	150			[PROSKE 2002]
E_x		1711	400			[SCHUEREMANS 2001]
E_x		1388	611			[SCHUEREMANS 2001]
E_x		1398	896			[SCHUEREMANS 2001]
E_y	normal	1673	497			[SCHUEREMANS 2001]
E_y		1690	684			[SCHUEREMANS 2001]
E_y	normal	10000	2500			[NOVÁK AND ZÁK 1997]
E_y	normal	5500	230			[NOVÁK AND ZÁK 1997]
E_y		8703	555			[GREIFENHAGEN 2001]
E_y		4113	412			[GREIFENHAGEN 2001]
E_y		9013	4336			[GREIFENHAGEN 2001]
E_y		5000				[TRINGALI ET AL. 2000]
G_x		883	271			[SCHUEREMANS 2001]
G_x		748	108			[SCHUEREMANS 2001]
η		0.37	0.12			[BINDA 2000]
η		0.20	0.06			[GREIFENHAGEN 2001]
η		0.12	0.11			[GREIFENHAGEN 2001]
Damping ratio				0.77	2.77	[RAMOS AND LOURENCO 2005]
Damping ratio				1.78	3.97	[RAMOS AND LOURENCO 2005]

*Flexural strength; Units in N/mm², Density in kg/m³

Table 4 - 5: Statistical values of material and structural parameters for masonry structures

Of course, the sources cited above represent only a small sample of the huge amounts of material tests, which were considered. Other – not so well documented – sources provide information of parameter values, which fall in the same range. Consequently, the values given in the previous tables are assumed to be sufficiently representative. Additional information could be gathered by relating the parameters by common equations, such as the relation of elastic and shear modulus written in equation 4.7.

$$G = \frac{E}{2 \cdot (1+\nu)} \tag{4.7}$$

The procedures to test the material properties of masonry are not always performed in the same layout. Such is that the shear strength of masonry may be determined in several different ways. Thus, the data presented were not always comparable, because the information about the type of test was seldom precise enough. However, the scatter and the type of distribution may be derived, which is the sole purpose of table 4-5. As the last remark, it should be noted that for practical purposes the difference between normal and lognormal distributions is negligible for coefficients of variation below 0.1 [BENJAMIN AND CORNELL 1970].

4.3.3 Additional parameters

For some of the parameters considered in the material model, sufficient information could not be found, because the sources were not well documented enough. Information is still missing for the softening behaviour and the two compliance parameters for brick and mortar respectively. Also, minimum and maximum values for the damping need to be given for the application of the numerical method. The finite element program uses Rayleigh damping, which is described by the mass damping coefficient α and a stiffness damping coefficient β. The overall damping may be determined for each frequency ω_i in dependence of those two factors as given in formula 4.8.

$$\varsigma_i = \frac{\alpha}{2 \cdot \omega_i} + \frac{\beta \cdot \omega_i}{2} \tag{4.8}$$

Constant factors over the necessary frequency range may be evaluated by formula 4.9 including the damping for the lowest (1) und highest frequency (m) and their corresponding damping included in the study.

$$\beta = \frac{2 \cdot \varsigma_1 \cdot \omega_1 - 2 \cdot \varsigma_m \cdot \omega_m}{\omega_1^2 - \omega_m^2} \tag{4.9}$$

Figure 4-15 shows the damping ratio for values given by α = 0.62 and β = 0.001 as well as the influence of mass damping, which is decreasing with increasing frequency and the importance of increasing stiffness or beta damping. The final values of α and β were chosen to represent a larger range of damping ratios for the frequency range between 0.5 and 10 Hz.

Concerning the softening parameters, figure 3-6 already explained that the possible range lies between 0, which would correspond to a perfectly plastic material and 1, which is almost completely brittle. Typical values according to [GAMBAROTTA AND LAGOMARSINO 1997] are about 0.8 for mortar and 0.4 for bricks. Since masonry is very brittle, it was assumed that the distributions of the softening parameters are triangular with upper values of 1.0 and lower value for 0.3 and the peak at the typical values already mentioned.

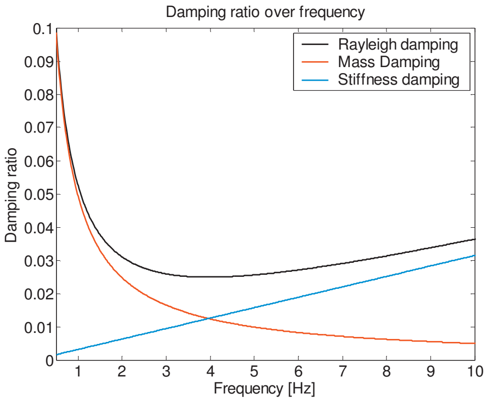


Figure 4 - 15: Rayleigh damping versus structural frequency

[GAMBAROTTA AND LAGOMARSINO 1997] serves as the only source – next to personal conversation – for the compliance parameters, which are assumed to be 1.0 at the mean and range between 0.5 and 1.5. All this information and consideration leads to the distribution of the parameters as presented in table 4-6. If not stated otherwise, these data are used in the calculations described in this chapter.

Parameter	Distribution	Mean	Dev.	Min.	Max.
μ	lognormal	0.6	0.111	-	-
σ_{mr}	lognormal	0.15 N/mm²	0.0525	-	-
τ_{mr}	lognormal	0.20 N/mm²	0.06	-	-
σ_{br}	lognormal	3.5 N/mm²	0.665	-	-
τ_{br}	lognormal	1.5 N/mm²	0.45	-	-
η	lognormal	0.1	0.025	-	-
ρ	normal	2000 kg/m³	150	-	-
E_x/E_y	normal	2000 N/mm²	240	-	-
β_m	triangular	0.8	-	0.3	1.0
β_b	triangular	0.4	-	0.3	1.0
c_{mt}	uniform	1.0	-	0.5	1.5
c_{bt}	uniform	1.0	-	0.5	1.5
adamp	uniform	0.62	-	0.3875	0.8215
bdamp	uniform	0.0004	-	0.0002	0.0006

Table 4 - 6: Parameters used and applied distributions

4.4 Numerical modelling

4.4.1 Introduction

To achieve reliable results a large number of calculations was performed. A complete overview will be given in table 4-7. But before, the type of elements included in the dynamic calculations will be presented. The elements were chosen to give representative samples of the macroelements listed in table 4-2. Special importance was laid on the fact that differences between vaults and wall elements could be analysed. All elements are based on real structures. Most were developed as smaller structural parts of the church presented in figure 4-12 and 4-13, the arch plotted in 4-6 may serve as another real example, other were generated according to information given in [AUGUSTI ET AL. 2001] or [LOURENCO AND OLIVIERA 2005]. All calculations were performed using the finite element program ANSYS®. A sample batch file, which explains the setup of files using the material model, is given in appendix F.

4.4.2 Walls

The simplest elements analysed were different configurations of walls. These were not only used to evaluate basic effects of the material parameters and the test calculations performed in chapter 3, but also to compare pushover to time history results.

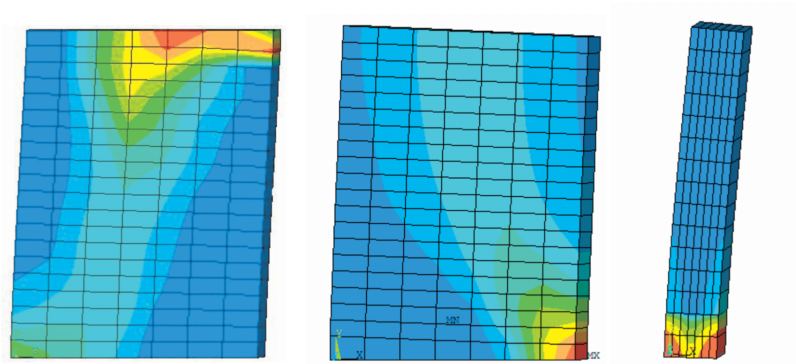


Figure 4 - 16: Some wall elements, damage in the mortar bed layer (left), brick damage (middle), mortar damage (right)

The results could be used for both, in-plane and out-of-plane mechanisms in the wall. While the two left models in figure 4-16 deal with in-plane mechanisms, the out-of-plane mechanisms were represented by models such as the one shown in figure 4-16 on the right. This model shows a cross section of a wall, assuming it is not restrained at the top. One configuration with top anchoring was also analysed. In contrast to figure 4-16, most models were constrained at the top, so that only a displacement was admitted, but no rotation.

4.4.3 Triumphal arch

The triumphal arch was another basic element analysed. It proved to give similar results to the walls, although of course the damage patterns were different. This element was also used for both, time history and pushover analysis. The results obtained can be well compared to [GIORDANO ET AL. 2001], resulting in the same damage mechanisms, but with a damage description which is more precise in location and probability.

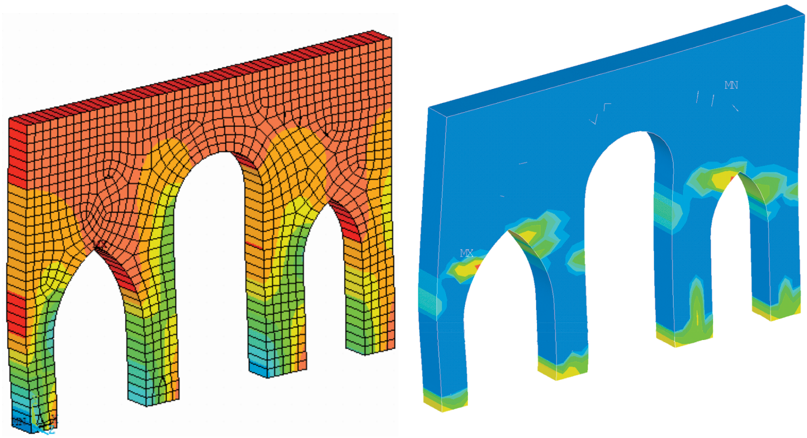


Figure 4 - 17: Triumphal arch, vertical stresses (left), accumulated mortar damage (right)

4.4.4 Vaults

Vaults proved to be far more difficult in modelling than the wall elements. Some of the analysed types of vaults are plotted in the picture below.

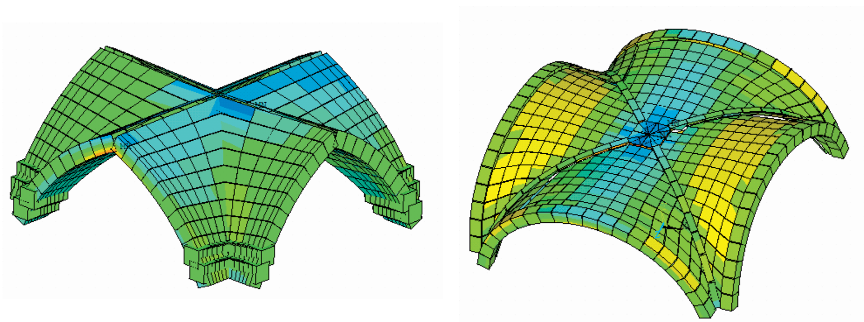


Figure 4 - 18: Different types of vault and horizontal stress states, blue colour is indicating compressive stresses

Different kinds of vaults included pointed arches, round arches, cross vaults or barrel vaults. Vaults were analysed for changing geometry, differential displacement at the base and additional mass elements representing the infill at the abutment, which can for example be seen in the vault plotted on the left side of figure 4-18. The geometrical changes included the thickness of the vault and the span between the abutments. Vaults generally failed more often numerically than the walls. This is ascribed on the one hand as a result of more sensible structures, especially with respect to the differential displacement at the abutments, but also as a drawback of the material model in case of insufficient modelling. Thus, the vault models had to be very elaborate and considered only geometrical boundary conditions which provided stable results for a greater number of simulations.

Additionally, different configurations were tested for arches, cf. figure 4-19. The alterations included for the vaults were also considered for the arches, such as the additional mass elements indicated by the blocks close to the foundation. These mass elements varied from not being existent until reaching to the top of the structure. For both structural types it proved to be more efficient to model the structure by layered shell elements than by the simple shells. In this way, a more realistic stress distribution over the thickness of the shell is achieved.

The time histories of the ground motions applied to the arches were random time histories. Additionally – as it was done for all vaults – the applied ground motion was filtered by a SDOF system to reflect the underlying structure. Various alternations of the structural frequency were included. Nevertheless, the data were not sufficient to analyse a significant contribution of the effects of periodic excitation at the base of the arched structures.

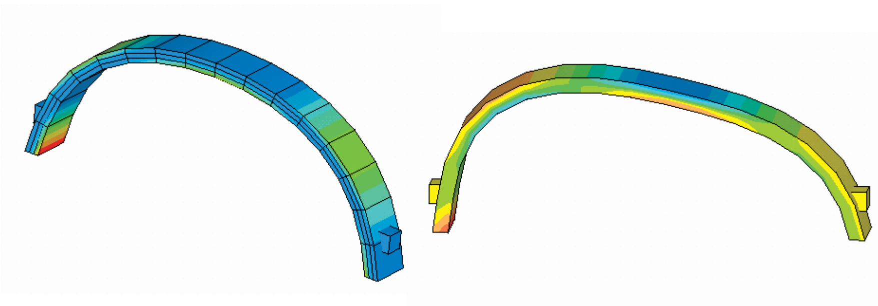


Figure 4 - 19: Analysed arch structure, mortar damage (left) and equivalent stresses (right)

4.4.5 Combined structural elements

Combined structural elements include all larger structures tested in numerical experiments. Basically, they may be subdivided into the tower and front part of the church, as explained in figure 4-20, a cross section of a nave, plotted in figure 4-21 and simulations of the abside. The tower was the only element which was tested under varying geometry including altering material parameters. The nave was only created once. In the beginning it was used to create reliable input data for the evaluation of the structural response of the vaults. Afterwards, it was used to check whether the results obtained in the analysis of single walls and vaults are transferable to larger structures. Concerning the abside, a full simulation proved difficult, especially with respect to the complex setup of the vaults and the intersection with other structural parts. It is thus recommended to divide the abside into single vaults and walls, which are then analysed.

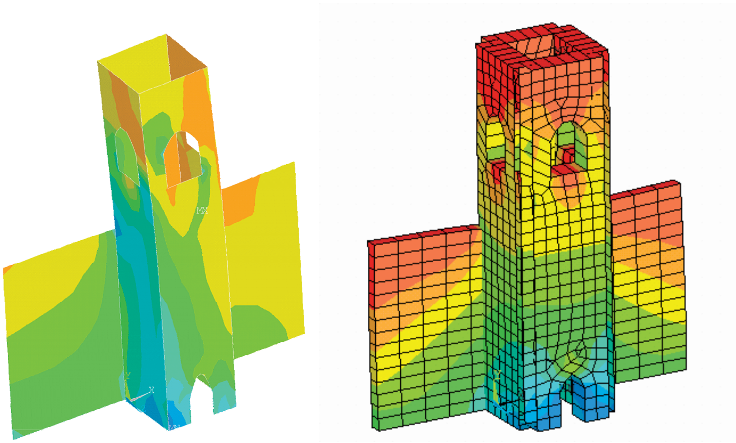


Figure 4 - 20: Tower configuration, vertical stresses in different excitation levels

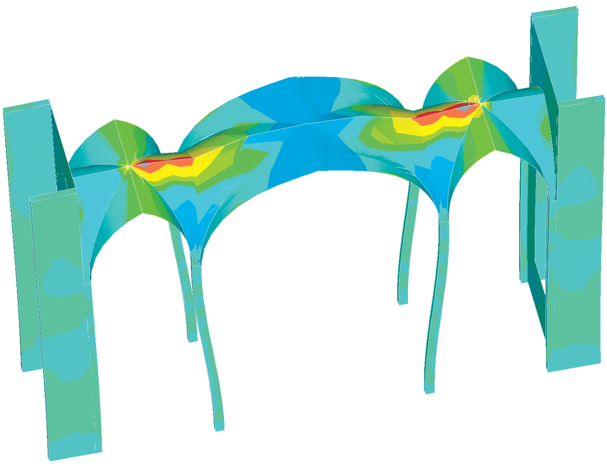


Figure 4 - 21: Mortar damage in the nave at the upper side of the vaults

4.4.6 Projections

Despite they are included in the macroelement scheme already presented, projections, e.g. finials, are normally not included in the vulnerability assessment, although their structural vulnerability may be determined rather easily. Two examples and the idealization of the middle one are shown in figure 4-22.

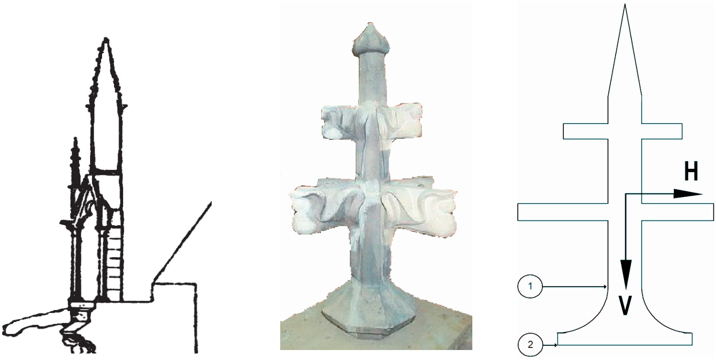


Figure 4 - 22: Two examples for finials and their idealization

The only type of failure which has to be considered for these types of elements is the so called rocking or overturning. This can be easily done, even with taking the tensile strength into account. Two possible cross sections have to be observed. The first is located in the lower part of the structure made out of stone with a usually rather low cross sectional area. This section is denoted with number 1 in figure 4-22. The second one is the cross section of the bottom of the finial, which is usually connected to the building with a mortar bed. Although this cross sectional area is significantly larger, the maximum tensile strength is far less. Now, the horizontal acceleration leading to cracking may be determined. With stresses in N/mm², geometry in mm units and weight in kg, the maximum horizontal acceleration in m/s² for which no damage occurs is given by formula 4.10. It assumes that the onset of cracking corresponds to structural collapse.

$$\ddot{x} \geq \frac{M_{crack}}{W \cdot h} = \frac{\dfrac{\sigma_t + \dfrac{V}{w_a \cdot w_b}}{6 \cdot w_a} \cdot w_a^{\,3} \cdot w_b}{W \cdot h} \qquad (4.10)$$

Where M_{crack} is the moment when cracking starts, i.e. the tensile strength is exceeded, σ_t being the tensile strength of stone in section 1 and that of mortar in section 2, w_a and w_b are the widths in the cross sectional plane. The collapse probability is then given by the probability of exceedance of the ground motion. Such simplifications have been excluded so far, but this is an example were detailed numerical modelling would not lead to improved results, since stiffness degradation effects and strength loss do not govern the behaviour of this type of elements.

4.5 Calculation overview

A wide range of models and different material properties have been explained so far, so that the complete number of calculation performed for this study is summarized in table 4-7. The elements correspond to the explanations made before. Additional information about the elements is given in Appendix G.

No.	Element	Number of simulations	Purpose
1	1	2000	Sensitivities and scatter for an scenario earthquake, $M_L = 4.0$, distance 15 kilometres, shear wall
2	1	2000	Sensitivities and scatter for an scenario earthquake, $M_L = 4.5$, distance 15 kilometres, shear wall
3	1	2000	Sensitivities and scatter for an scenario earthquake, $M_L = 5.0$, distance 15 kilometres, shear wall
4	1	2000	Sensitivities and scatter for an scenario earthquake, $M_L = 5.5$, distance 15 kilometres, shear wall
5	1	2000	Sensitivities and scatter for an scenario earthquake, $M_L = 6.0$, distance 15 kilometres, shear wall
6	1	2000	Sensitivities and scatter for an scenario earthquake, $M_L = 6.5$, distance 15 kilometres, shear wall
7	1	2000	Sensitivities and scatter for an scenario earthquake, $M_L = 7.0$, distance 15 kilometres, shear wall
8	1	10000	Sensitivities and distributions shear wall *
9	1	5x1000	Sensitivities and distributions of other material distributions, shear wall
10	2	10000	Influence of the height/width ratio, shear wall
11	3	1000	Sensitivities and scatter for triumphal arch
12	4	5000	Sensitivities and scatter for a barrel vault*
13	4	1000	Sensitivities and scatter for a barrel vault
14	5	5000	Sensitivities and scatter for an arch *
15	5	1000	Sensitivities and scatter for an arch
16	6	2000	Sensitivities and scatter for vault type 1, pointed vault*
17	6	1000	Sensitivities and scatter for vault type 1, pointed vault
18	7	1000	Sensitivities and scatter for vault type 2, arched vault*
19	7	1000	Sensitivities and scatter for vault type 2, arched vault
20	7	5x1000	Sensitivities and distributions of other material distributions, vault type 2
21	8	10	Sensitivities and scatter for cross section of nave
22	9	1000	Sensitivities and scatter for cross section of tower
23	10	1000	Sensitivities and scatter for out-of-plane, shear wall
24	11	1000	Sensitivities and scatter for out-of-plane, shear wall with top anchoring

Table 4 - 7: Overview of all calculations performed, an * denotes the calculations where the scatter of ground motions was included

4.6 Results

4.6.1 Remarks

The large amount of simulations and the diversity of the constructions analysed required a structured approach to assess the influence of each parameter. Consequently, the first step included only the general assessment of the sensitivities by varying only one parameter. In a second step, the variability of the output parameters was assessed by including the full probability distributions for the material and structural parameters as shown in table 4-6. Before the influence of the ground motions was also included, several levels of ground motions were tested to see, whether the results are valid for different levels of ground motions. Finally, full analyses including all material, geometrical and load parameters were carried out. The results will be explained step by step.

4.6.2 First sensitivity tests

The first simulations included only the variation of one parameter. In most cases, the values of ±2 and ±1 standard deviations were taken. Otherwise, minimum or maximum values were chosen. The complete list of the variations of the material parameters is shown in the following table. For all other parameters, which were not varied, the mean values listed in table 4-6 were taken. The time history applied is the one plotted in figure 3-5 and the structure similar to the wall described in figure 4-16 with a width of two metres and a height of three metres.

Parameter	Number	Unit	Val. 1	Val. 2	Val. 3	Val. 4
μ	1		0.3	0.49	0.71	0.9
σ_{mr}	2	N/mm²	0.06	0.095	0.205	0.26
τ_{mr}	3	N/mm²	0.09	0.14	0.26	0.4
c_{mt}	4		0.5	0.8	1.7	2.0
β_m	5		0.3	0.55	0.85	1.0
σ_{br}	6	N/mm²	1.3	2.8	4.2	5.1
τ_{br}	7	N/mm²	0.7	1.05	1.95	3.0
c_{bt}	8		0.5	0.8	1.7	2.0
β_b	9		0.3	0.42	0.71	1.0
E	10	N/mm²	1300	1650	2350	2700
η	11		0.04	0.075	0.125	0.16
adamp	12		0.3785	0.48	0.72	0.8215
bdamp	13		0.0002	0.0003	0.0005	0.0006
ρ	14	kg/m³	1400	1760	2240	2600

Table 4 - 8: Variations of the considered input parameters

In table 4-8, each parameter is assigned a number, which is used as a reference in figures 4-23 to 4-29. Figure 4-23 explains the impact of the variables on the maximum relative displacement at the top of structure.

A wide range of variables seems to have an impact on the displacement. Damping - mass and stiffness in equal proportions - elastic modulus and density contribute at most to the structural response. Interestingly, the mortar properties show also a large influence on the results, whereas reflecting the brick properties, only the compressive strength has an influence on the result.

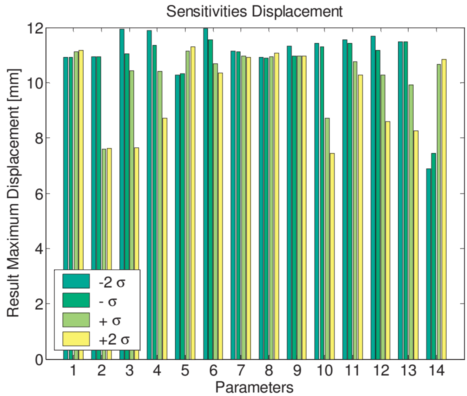


Figure 4 - 23: Influence of the input parameters on the maximum relative displacement

What is also important to realize is that while the mean displacement using the mean values of table 4-6 is close to 10.5 mm, in some cases the mean of the four simulations performed is below 10.5 mm. This is a result of the complex material behaviour. Regarding the mortar tensile strength, i.e. number 2, cracking occurs if the mean value is taken. No cracking occurs, if the tensile strength is increased, resulting in values significantly lower values of the top displacement. On the other hand, a further decrease of the tensile strength does not result in a much higher displacement, because first cracks have already occurred and the structural stiffness decreased. Similar conclusions can be drawn for the other parameters considered in figure 4-23. Altogether it cannot be expected that the mean values of all 4 simulations fit the overall mean. This will be encountered again in the analysis of the following output parameters.

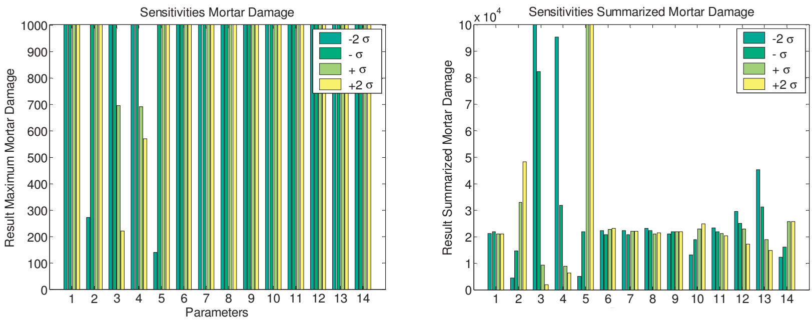


Figure 4 - 24: Influence of the input parameters on the mortar damage

The local mortar damage was harder to analyse, because very high values were achieved. Again, the maximum value was set to 1000 because numbers above this value do not exhibit significant influence on the structural behaviour any more. Instead, the global mortar damage accumulated over the whole structure provides better information about the correlations.

As it would be expected, the mortar material parameters show the highest correlation with the damage in the mortar layer. As shear strength (parameter 3) and inelastic compliance (parameter 4) of the mortar increase, the damage suffered by the mortar layer rapidly decreases. On the other hand, as the material gets more brittle, denoted by parameter 5, damage increases. Only the tensile strength of the masonry (parameter 2) is reacting somewhat different. As it increases, the mortar damage increases as well. This seems very odd at the first sight. The same effect was observed in [GAMBAROTTA AND LAGOMARSINO 1997]. Higher mortar tensile strength leads to higher local damage concentrating on a smaller number of locations, whereas low strength mortar results in a smeared global damage. Because of extremely high values of local damage for high strength mortar, the summation of all damages over the structure results in higher values, too – although the general damage is less. Results must thus be interpreted carefully. Although it can be said that the damage is more localized and higher damage values are obtained, this does not necessarily mean that the complete structure suffers greater damage. This is underlined also by figure 3-6, which shows no significant loss of toughness for values of the damage parameter above 100. Another reason is that, if the mortar has a higher tensile strength, it is not destroyed as easily. Thus, it is also capable of transferring more shear stresses, leading in turn to higher damage variables. Of all other parameters, damping, elastic modulus and density show some effect. This influence can be observed for all other parameters as well, as the following figures will show.

Throughout the calculations the bricks suffered only minor damage, thus the only brick material parameter showing a remarkable dependence on the brick damage is the compressive strength. Naturally, the density is the main parameter influencing the vertical stresses, which leads to cracks in the bricks. Additionally, the mass distribution determines the dynamic forces resulting from the ground excitation. They are increasing as the density becomes higher. Therefore, this parameter is also governing the results clearly. Mortar parameters show some dependence because they influence the structural behaviour massively and determine the stress occurrence in the structure. Some slight differences between global and local effects may be observed, such as the increasing local damage with increasing elastic modulus (parameter 10) or the decreasing local brick damage as the mortar gets more brittle (parameter 5). Both effects are not evident for the global damage. The reason for this could not be clearly identified, generally the local damage reacts more sensible, whereas global damage is more smeared and influenced by a smaller number of parameters.

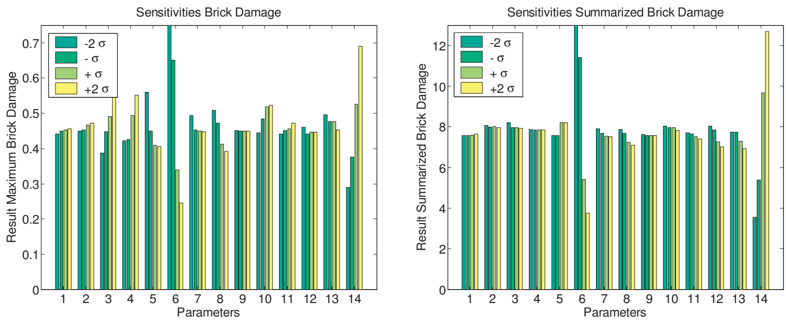


Figure 4 - 25: Influence of the input parameters on the brick damage

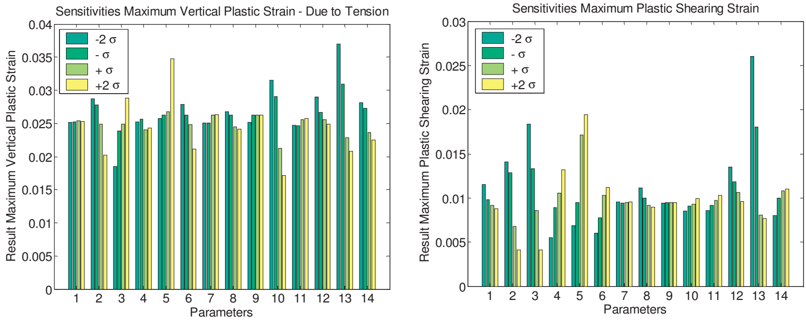


Figure 4 - 26: Influence of the input parameters on the strains

The results for the plastic strains, which are plotted in figure 4-26, are as it would be expected. They are governed by the mortar input variables, the elastic modulus and the stiffness damping, which plays also an important role for the other output parameters. The vertical plastic strains are those occurring due to tension in the mortar plane, thus an increasing density leads to lower strains.

Concerning the stresses, results resemble also those which would have been estimated before the analysis. The mortar variables prove once more that they are of uttermost importance for the correct description of the masonry material. Regarding the equivalent stresses it is Young's modulus and the density, again together with the mortar parameters, which largely influence the computational results. The same is true for the shear stresses plotted in figure 4-28.

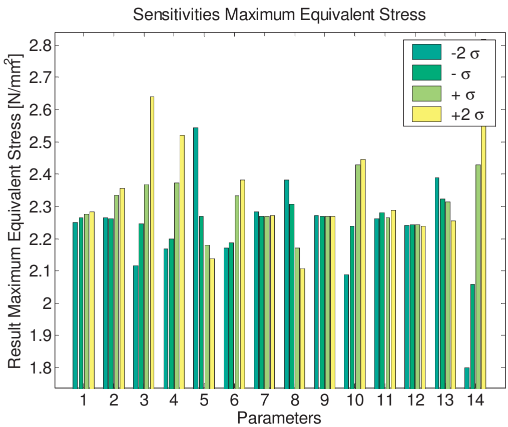


Figure 4 - 27: Influence of the input parameters on the equivalent stress

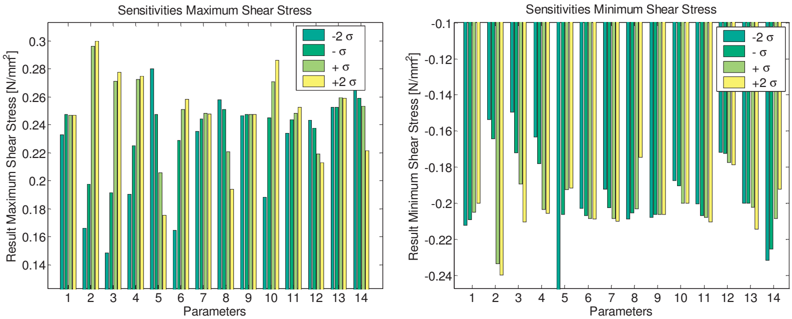


Figure 4 - 28: Influence of the input parameters on the shear stress

Concerning the vertical stresses that may occur, it must be distinguished between the maximum compressive stresses, which are shown on the left side and the maximum tensile stresses, depicted on the right of figure 4-29. For the tensile stresses it can be concluded that the sole parameter influencing the structural result is the tensile strength of the masonry. On the other hand, the occurrence of vertical compressive stresses is regulated by a larger number of factors. Again, it is the properties of the mortar, as well as the Young's modulus and – most dominant – the density of the structure, which are contributing significantly to the results.

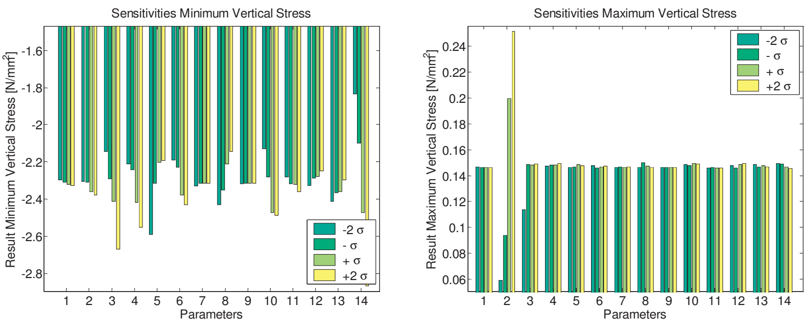


Figure 4 - 29: Influence of the input parameters on the vertical stress

In conclusion, two groups may be identified that govern the structural behaviour. At first, response is governed largely by density, Young's modulus and structural damping, which determine the elastic response. Next to these effects, the mortar properties show the highest influence on the results, emphasizing their importance for the description of the material and the nonlinear behaviour.

4.6.3 Scatter and sensitivities for a fixed ground motion

4.6.3.1 Output parameter sensitivities

The first sample test were performed to understand the structural behaviour and to cross check the results obtained by further analyses. To obtain additional information, especially about the type of distribution of the output parameters, but also to receive a more detailed insight into the sensitivities about 100.000 nonlinear dynamic calculations for walls and roundabout 20.000 calculations for vaults and cupolas were performed. Some first results of these analyses were already presented in [URBAN ET AL. 2006] or [PEIL AND URBAN 2006]. The analysis was at first performed only by varying the material parameters while applying always the identical ground motion record.

Results for the sensitivities verified the influence of the material parameters analysed in the previous chapter. Giving the rank order correlation coefficients in brackets, it can be concluded that the displacement is governed mostly by the parameters elastic modulus (0.5) and density (0.35). Secondly, mortar material parameters (0.3) and damping contribute (0.25) mostly to the final result. If all variables with a correlation coefficient over 0.2 are taken into account, summarized and each divided by the total sum of the coefficients, the contribution towards the final results is given as follows. Concerning the top displacement of a wall, figure 4-30 plots the results for all computations done for walls.

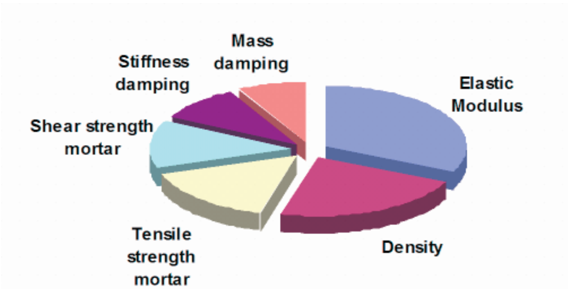
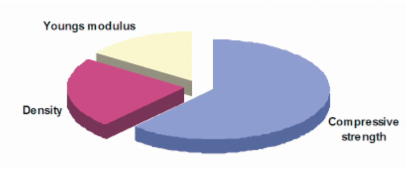


Figure 4 - 30: Contribution of input parameters to the final top displacement

Results for the two internal damage parameters also resemble those of the sensitivity test closely. The mortar damage is mostly influenced by the shear strength (0.6) and the tensile strength (0.4) of the mortar. The softening coefficient for the mortar is correlated with a coefficient of 0.4 with the local and up to 0.6 with the global mortar damage. Inelastic compliance is given by correlations around 0.4.

The damage in the bricks is simply determined by three parameters. The compressive strength is correlated with a factor of 0.8 with the occurring damage. Small contributions are added by density (0.3) and Young's modulus (0.2). This is true for both local and global brick damage. Figure 4-31 on the left visualizes the contributions of the material parameters.

Reflecting the previous conclusions, the strains are not easy to evaluate, as figure 4-31 on the right indicates. All considered strains are influenced by the parameters elastic modulus, density and the properties of mortar. The vertical strains are also slightly influenced by the stiffness and mass damping. All are correlated with factors around 0.3, apart from the shear strength of mortar which contributes most significantly to the overall shearing strains. Still, the results of the previous chapter could be repeated.



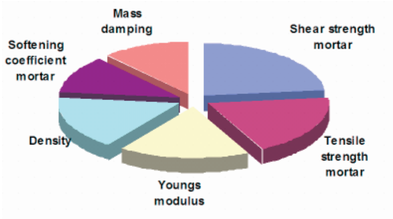


Figure 4 - 31: Contribution of input to brick damage (left) and shearing strains (right)

Results for the stresses were also similar to those of the previous analyses. For this reason only the correlation coefficients for the major parameters are listed in table 4-9. The results obtained are valid for walls and vaults, although the vaults show a higher correlation towards the material properties. Additional information is given in appendix G.

Output\Input	Mortar tensile strength	Mortar shear strength	Mortar softening	Compressive strength bricks	Density	Young's modulus	Mass damping	Stiffness damping
Equivalent Stress	0.099	0.156	-0.191	0.172	0.502	0.077	-0.120	-0.096
Shear stress	-0.414	0.366	-0.128	-0.017	-0.222	-0.054	0.081	0.213
Vertical compressive stress	-0.091	-0.171	0.199	-0.172	-0.506	-0.078	0.118	0.090
Vertical tension stress	0.969	-0.016	0.027	-0.029	-0.042	0.006	0.031	-0.013

Table 4 - 9: Rank order correlation coefficients for stress

4.6.3.2 Output parameter distributions

Parameter	Distribution type
Equivalent Stress	Normal /Lognormal
Shear Stress	Lognormal
Vertical compressive stress	Lognormal
Vertical tensile stress	Lognormal
Horizontal displacement	Lognormal
Localized Mortar Damage	Exponential
Global Mortar Damage	Lognormal/Exponential
Localized Brick Damage	Exponential
Global Brick Damage	Lognormal/Exponential
Local Vertical Strains	Lognormal/Exponential
Local Shearing Strains	Lognormal/Exponential

Table 4 - 10: Distributions for the output parameters

Next to the assessment of the sensitivities of the different material parameters it is important to calculate the scatter of the results and the type of distribution the output parameters exhibit. The results are listed in table 4-10. The types of distributions are assigned according to the least occurring error, which was determined by the least square method.

4.6.4 Dependence on the ground motion level

The intensity of the ground motion is commonly assumed to have the largest effect on the structural damage. Thus, before the effects of the scatter of the earthquake intensity were analysed, the scatter of the output related to six different levels of ground motions was computed. Therefore, artificial accelerograms were generated based on the algorithm of [SABETTA AND PUGLIESE 1996], cf. chapter 3, assuming the distance to the rupture would be 15 kilometres. Six different magnitudes were chosen and the following accelerograms generated.

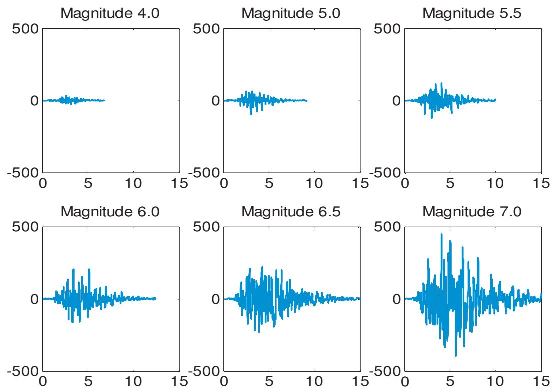


Figure 4 - 32: Generated accelerograms for the assessment of the influence of the ground motion level, the Y-axis gives the PGA in cm/s² while on the horizontal axis the time in seconds is plotted

The change in the intensity of ground motion did not cause a significant effect on the type of probability distribution, apart from the increase of mean values and a generally higher scatter of the results. The parameters and their sensitivities explained in the previous chapter are in most cases still valid. What could be observed though is that the importance of some parameters changes with the amplitude of the ground motion. In detail, they exhibit slight differences which are shortly explained in the following.

The two damping parameters show a small influence on the maximum top displacement for low excitations. Their importance increases with increasing intensity of the ground motion. A maximum correlation coefficient of 0.5 was calculated for the earthquake with a surface magnitude of 7.0. For more intense ground motions the displacement is furthermore influenced by the tensile and shear strength of the mortar joints and the softening coefficient of mortar, which do not contribute at all for very low intensities. The total scatter of possible maximum displacement with respect to altering material properties is plotted in the next figure.

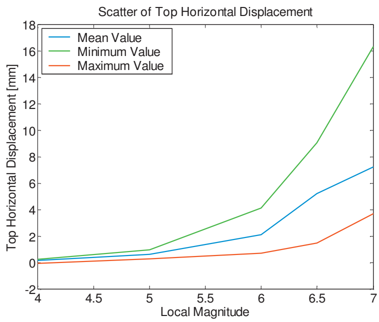


Figure 4 - 33: Scatter of displacement with increasing earthquake intensity

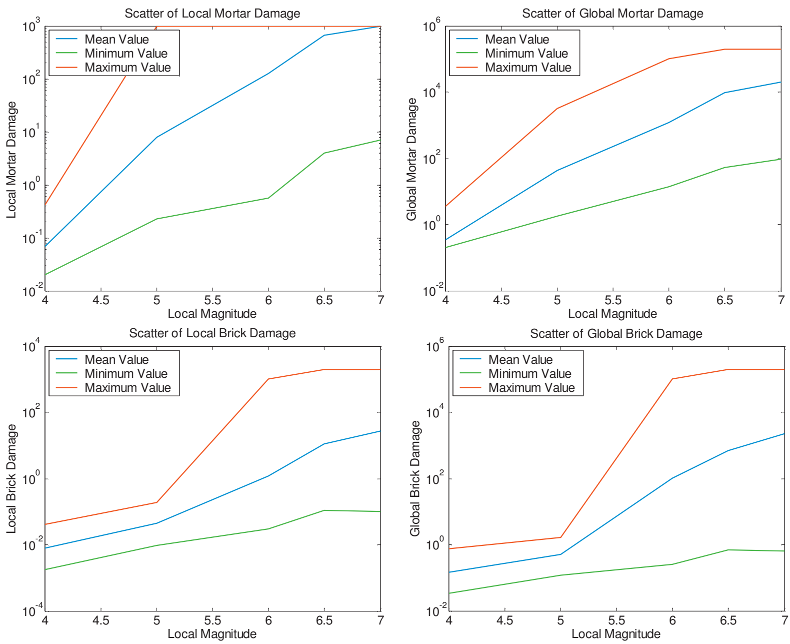


Figure 4 - 34: Scatter of the internal damage parameters

The scatter of both damage parameters in a global and a local scale is shown in the previous figure. Maximum values are flattening, because on a local scale the maximum value admitted was 1000 and on the global scale 100 000 as explained before. Otherwise they would further increase exponentially.

Stresses show better correlations for high ground motions. Density and elastic modulus are the highest contributors. Interestingly, the influence of both damping parameters on the stresses is decreasing with increasing strength of earthquake. Also, vertical stresses show clear dependence on a smaller number of input parameters for strong ground motions. Other parameters do not vary for different levels of ground motions. Some results are plotted in figures 4-35 and 4-36.

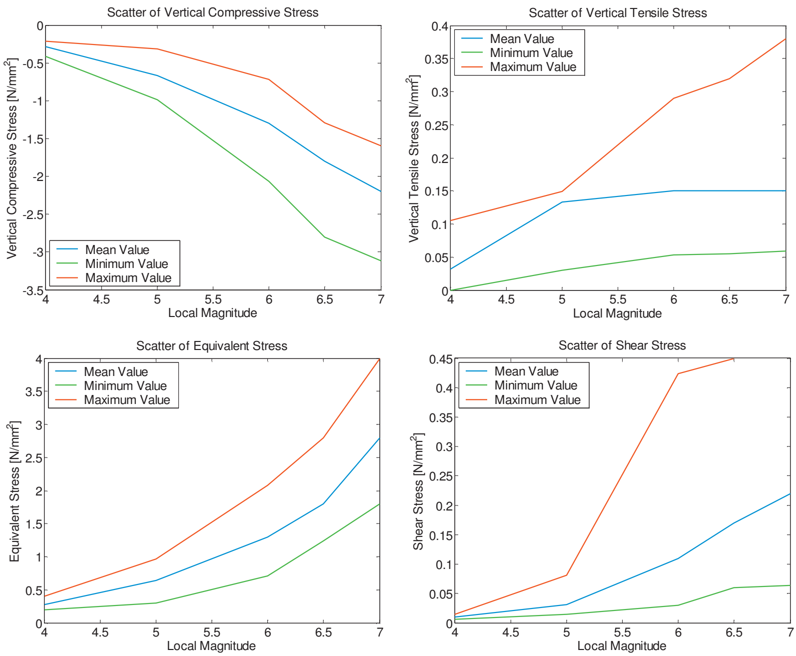


Figure 4 - 35: Scatter of maximum stresses

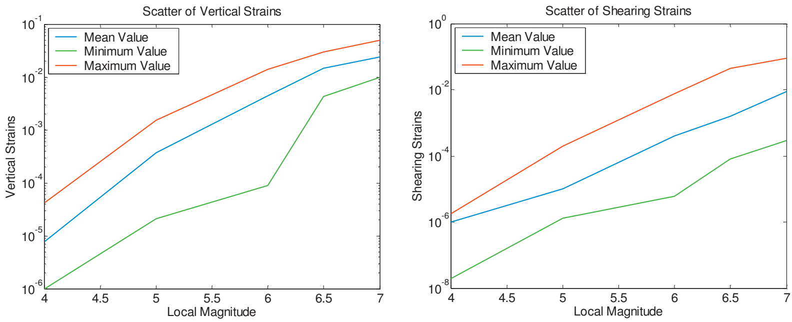


Figure 4 - 36: Scatter of maximum local strains

4.6.5 Including the variability of ground motion

4.6.5.1 Output parameter sensitivities

The next step performed is the consideration of the variability in the ground motion. Therefore, the horizontal and vertical acceleration were distributed after figure 3-23, using the attenuation of [SPUDICH ET AL. 1999], which returned an exponentially distributed probability of exceedance. It is once more underlined that this chapter includes only the variabilities of the PGA, although other parameters were determined as better correlated with the structural damage. This was done, because the calculations of this chapter and chapter 3 were performed in parallel. Because of this, it was not possible to include other effects as well. If the accelerations are also included in the calculations, it can easily be seen that it is the dominating parameter for the determination of most output parameters, if walls are analysed. The following table gives an overview about the influence of the acceleration for walls.

Parameter	**Influence acceleration**	**Corr. Coeff.**	**Second important parameter**	**Corr. Coeff.**
Equivalent stress	Vertical	0.6	Density	0.6
Shear stress	Horizontal	0.8	Density*	0.3*
Vertical compressive stress	Horizontal	0.7	Density*	0.3*
Displacement	Horizontal	0.9	Density	0.1
Brick Damage	Horizontal	0.75	Compressive strength	0.35
Mortar Damage	Horizontal	0.7	Shear strength mortar	0.25
Vertical Strains	Horizontal	0.8	Elastic modulus	0.25
Shearing Strains	Horizontal	0.8	Shear strength mortar	0.3

* In these cases the elastic modulus had also a coefficient of 0.3

Table 4 - 11: Correlation coefficients for acceleration and second dominant parameter for walls

The results for vaults differ significantly compared to the results of the sensitivities analyses for walls. In all simulations of vaulted structures, the horizontal acceleration has proven important only for arches and barrel vaults, where the highest correlation coefficient of 0.5 occurred. Instead, the material parameters and the vertical acceleration are the dominating parameters for cross vaults. Another important parameter, which had a large influence on the results of the numerical simulation, was the static displacement applied before the strong motion. The static displacement was included to take into account the different displacement of pillars and walls caused by settlements. Next to the static displacement, the differential displacement reflects the different stiffness of the pillars in comparison to the outer walls of a structure, which results in different excitations of the base of the vaults during the earthquake. The contributions of the input parameters towards the final result of the mortar damage are plotted in figure 4-37. This picture also compares the effects to those of brick damage in a wall. A complete overview for the most important output parameters is given in the following table; additional information is given in appendix G.

Parameter	Input parameter	Corr. Coeff.	Input parameter	Corr. Coeff.	Input parameter	Corr. Coeff.
Equivalent stress	Static displacement	0.4	Elastic modulus	0.4	Vertical acceleration	0.3
Shear stress	Static displacement	0.5	Elastic modulus, Differential displ.	0.2	Shear strength	0.15
Vertical compressive stress	Static displacement	0.5	Horizontal acceleration	0.2	Shear strength, Elastic modulus	0.2
Displacement	Horizontal acceleration	0.85	Static displacement	0.7	Damping	0.2
Brick Damage	Compressive Strength	0.7	Static displacement, Differential displ.	0.4	Softening, Damping	0.2
Mortar Damage	Shear Strength	0.65	Static displacement, Differential displ.	0.4	Vertical acceleration	0.3
Vertical Strains	Horizontal acceleration	0.4	Shear strength	0.3	Tensile strength, Static displacement	0.2
Shearing Strains	Shear strength	0.3	Static displacement	0.2	Horizontal acceleration	0.2

Table 4 - 12: Correlation coefficient for dominant parameters in vaults

It was also tried to analyse the effect of the filtering of ground motions by the substructure. In some cases, a correlation between the frequency of the substructure and the damage was observed with a coefficient up to 0.3. However, these results are neither as important as the influence of other parameters, nor were sufficient calculations performed to provide reliable results.

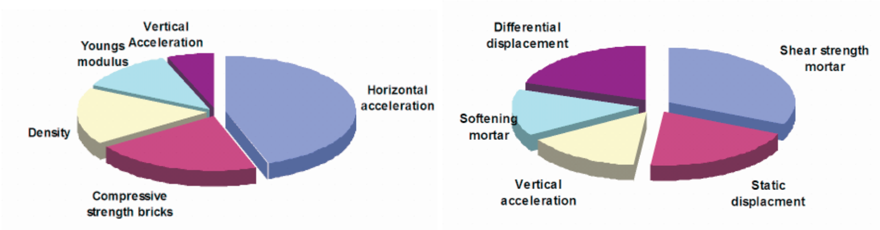


Figure 4 - 37: Comparison of important parameters for brick damage in walls (left) and mortar damage in vaults (right)

4.6.5.2 Output parameter distribution

Finally, the distributions of the output parameters including the strong motion have to be presented. The types of distributions without the ground motion were already presented in table 4-10. Despite it would be expected that they might change, if the variability of the earthquakes is included, no significant change in the type of distribution could be observed. Instead, the characteristics are even more intense and easier to recognize. The distributions are independent of the structural type as figures 4-38 and 4-39 show.

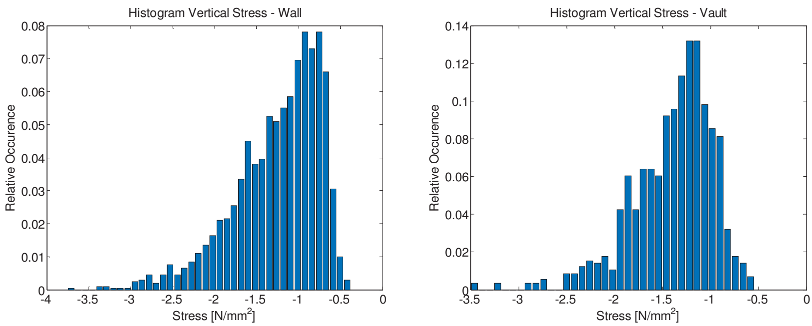


Figure 4 - 38: Distribution of vertical stresses in walls (left) and vaults (right)

If the material properties change in their statistical values while still exhibiting the same kind of probability distribution, the mean of the output distribution is shifted, but the shape remains the same. General remarks, which can be given about the type of distribution of the output parameters, are that all types of stresses are distributed in a lognormal manner. The distributions for the damage variables exhibit more an exponential type of distribution. In the applied material model the strains are linearly dependent on the stresses plus an inelastic contribution governed by the damage variables. Thus, for moderate events without high stress occurrence, the strains are lognormal distributed, whereas for higher ground accelerations, the inelastic contribution governs the phenomenon and strains are also exponentially distributed.

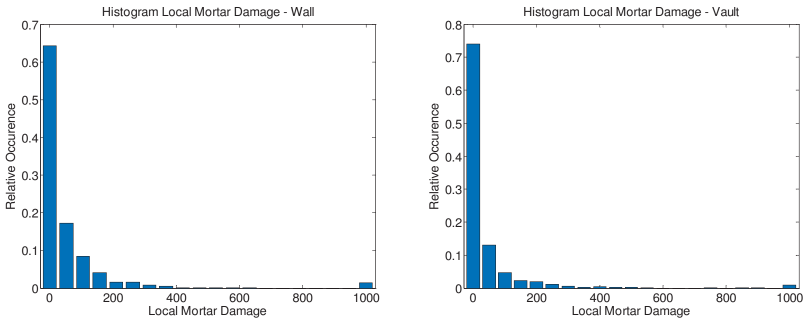


Figure 4 - 39: Distribution of local mortar damage in walls (left) and vaults (right)

4.6.6 Influence of the width/height ratio

The influence of the width/height ratio of the walls is shown in figure 4-40. As can be seen, both scatter and absolute values increase as the ratio increases. It is expected that the results would be similar, if the influence of the span width to arch height for vaults would be considered. This is not analysed further, but it will be valuable information for the development of fast survey methods as the very basic one proposed by [LOURENCO AND OLIVIERA 2005] aimed at fast screening of churches with high vulnerability.

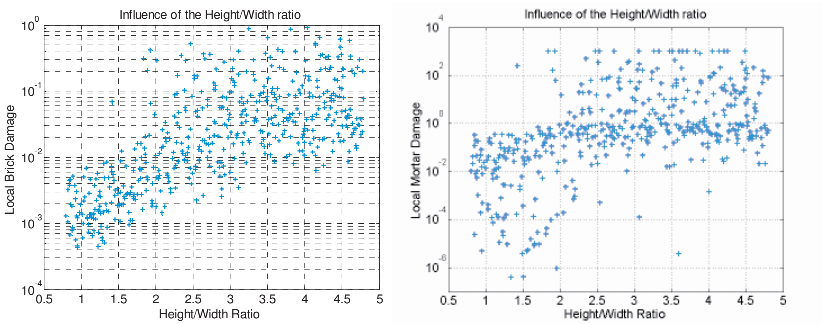


Figure 4 - 40: Influence of the height to width ratio on the scatter of local brick and mortar damage for walls

4.7 Critical review

4.7.1 Use of the results

A large number of simulations has been performed to assess the reliability of deterministic studies and to predict the scatter of the results. The effects were cross-checked for different structural configurations and altering statistical moments of the material parameters. The data presented are valid for the full range of observations. Significant differences were found and explained for arched structures and walls. Although in special applications the type of distributions used to describe the variability of material properties might differ, the results are applicable for most purposes and will enable engineers to present results more precise than it was possible ever before. The full application, using a series of deterministic tests will be visualized in chapter 7 and is not explained further at this point, because additional information need to be given in chapters 5 and 6.

The results show a large dependence on the material parameters, indicating that the reliability of most of the deterministic studies is similar to rough screening methods. Also, it becomes clear, how important a structural anamnesis is. For this, the data may provide important information about the setup of testing campaigns. Significant insights into the structural behaviour under earthquake excitations could be obtained. This was achieved by determination of the most important material parameters, by sensitivity analysis and Monte-Carlo simulations. These proved to be very helpful in performing the tasks of evaluating, relating and comparing the influences the model input parameters on the outcome of the calculations. Computations are and will remain complicated throughout this work. There is an urgent need for more simplified and reliable models. The results of this study may be used to assess the impact uncertainties have on simpler models. Furthermore, histograms of the occurrence of output parameters were obtained, which will be used in the next chapter to determine the probability of a certain damage grade.

4.7.2 Remarks on the model uncertainty

In reliability engineering, the model uncertainty is included by increasing the load factors. This cannot be realized in this study, because it would alter the results of the risk assessment. Nevertheless, it is a common opinion among risk analysts to provide additional information on aleatory variability and epistemic uncertainty. Thus, the type of uncertainty should always be presented as part of the analysis results. So far, no attempt has been made to qualify and quantify model uncertainties for earthquake excited masonry structures. This was largely because it was not possible to assess effects of aleatory variability at first. This drawback has at least to some extent been overcome by the data presented in this chapter.

The model uncertainty in the simulations was tried to keep as low as possible, by the use of a convenient material model and the introduction of sophisticated structural models. Although the model uncertainty still exists, the details of the models state a significant improvement as compared to static or linear dynamic models. If the model uncertainty has to be included, the approach present by [JCSS 2000] is recommended. This approach takes the outcome of the results to be the mean of a lognormal distribution with a coefficient of variation of 0.2.

4.8 Additional remarks

4.8.1 Modelling and computations of historical masonry structures

Throughout this work a large number of problems have been considered and several different calculations ranging from the static assessment of a triumphal arch to the dynamic analysis of full churches have been performed. Not all of them could be mentioned and explained. Still, some hopefully very fruitful remarks concerning computations on historical masonry structures can be made.

All models consisted of continuum macro-modelled structures. Macro-modelling in this case means that there was no necessity to differentiate between a mortar layer and brick layer. The applied material model describes the tensional and shearing behaviour by the mortar layer and the compressive behaviour by the material properties of the bricks. In this way, it is possible to derive damage output and material input parameters for both layers, although the model describes a continuum. For the applied structures and problems this was absolutely sufficient. It is not so much the problem of computation time, but of man hours needed to create an accurate structural model. Also, the results showed that it is absolutely necessary to use models that are able to describe the strength and stiffness degradation of masonry, while they also revealed clearly that it is not sufficient to rely merely on elastic ideal-plastic models, since they do not reflect the degradation of the material. The prediction of the post peak response of the masonry block is the most critical task when addressing structural safety.

An attempt was made to perform a nonlinear dynamic calculation of a full church. This proved to be practically impossible, since many element formulations would require correct assumption of coupling the degrees of freedom as well as the orientation of the mortar layers throughout the full construction. Also, this approach would have provided results only for one specific deterministic study.

If vaults are modelled, the elements should not consist of several layers, which can be done in ANSYS® for example by using shell 91 elements. Otherwise a correct stress distribution is hardly possible and results may be erroneous. The use of pure linear elastic calculations is to be avoided in seismic risk analysis. Similar results have been obtained by [GREIFENHAGEN 2001].

Given the difficulties explained throughout the text, it is obvious that the description of masonry still consists of strong simplifications, although most of them were tried to be eliminated here. Assessing the seismic risk is not a task which can be easily performed within a couple of hours and will probably be performed only by experts during the next years. On the other hand, this work is showing that with reasonable effort very good predictions of the seismic risk and probabilities of failures can be made.

4.8.2 Assessing the probability of failure

Finally, how do all these results contribute to the assessment of the probability of failure? In the simplest way, collapse can be seen as numerical failure or the non-convergence of the iteration. This occurred as often as it is explained corresponding to table 4-13.

No.	Element	Number of simulations	Purpose	Numerical failure collapse
1	1	2000	Sensitivities and scatter for an scenario earthquake, $M_L = 4.0$, distance 15 kilometres, shear wall	0
2	1	2000	Sensitivities and scatter for an scenario earthquake, $M_L = 4.5$, distance 15 kilometres, shear wall	0
3	1	2000	Sensitivities and scatter for an scenario earthquake, $M_L = 5.0$, distance 15 kilometres, shear wall	0
4	1	2000	Sensitivities and scatter for an scenario earthquake, $M_L = 5.5$, distance 15 kilometres, shear wall	1
5	1	2000	Sensitivities and scatter for an scenario earthquake, $M_L = 6.0$, distance 15 kilometres, shear wall	8
6	1	2000	Sensitivities and scatter for an scenario earthquake, $M_L = 6.5$, distance 15 kilometres, shear wall	18
7	1	2000	Sensitivities and scatter for an scenario earthquake, $M_L = 7.0$, distance 15 kilometres, shear wall	31
8	1	10000	Sensitivities and distributions shear wall *	5
9	1	5x1000	Sensitivities and distributions of other material distributions, shear wall	2
10	2	10000	Influence of the height/width ratio, shear wall	10
11	3	1000	Sensitivities and scatter for triumphal arch*	2
12	4	5000	Sensitivities and scatter for a barrel vault*	3
13	4	1000	Sensitivities and scatter for a barrel vault	7
14	5	5000	Sensitivities and scatter for an arch *	2
15	5	1000	Sensitivities and scatter for an arch	6
16	6	2000	Sensitivities and scatter for vault type 1, pointed vault*	15
17	6	1000	Sensitivities and scatter for vault type 1, pointed vault	24
18	7	1000	Sensitivities and scatter for vault type 2, arched vault*	12
19	7	1000	Sensitivities and scatter for vault type 2, arched vault	20
20	7	5x1000	Sensitivities and distributions of other material distributions, vault type 2	23
21	8	10	Sensitivities and scatter for cross section of nave	1
22	9	1000	Sensitivities and scatter for cross section of tower	3
23	10	1000	Sensitivities and scatter for out-of-plane, shear wall	12
24	11	1000	Sensitivities and scatter for out-of-plane, shear wall with top anchoring	3

Table 4 - 13: Number of numerical failures in the simulations performed

Certainly, a more precise approach is necessary. To this end, the next chapter will provide additional information and chapter 7 will finally explain, how the data can be used in assessing both, the damage grade and the probability of failure.

Chapter 5

Damage description

5.1 Introduction

The results of the previous chapter allow engineers to express the numerical results in probabilistic terms. To predict structural damage and resulting losses, it is necessary to relate the numerical output to discrete damage grades. Therefore, it is necessary to define practicable discrete damage grades and constant damage indices which are then correlated to discrete partial damages. Both, existing damage grades and existing damage indices will be discussed in the following text and conclusions for their application on this topic drawn. Finally, an approach is presented, which explains how the numerical results may be connected to the observed structural damage. With this advancement it will also be possible to express the probability of damage occurrence.

5.2 Review of damage intensity scales

The application of damage scales is to some extent ambiguous. Although on the one hand a simple damage scale is required, which can be easily applied, detailed information has to be included on the other hand. Thus, assigning damage grades or damage rates, even if based on numerical experiments, will require some subjective assessment and be a compromise of the two aims mentioned above. With respect to masonry structures, several types of damages might be imagined. Between the onset of cracking and the occurrence of visual cracks, the fall down of pieces of plaster up to the complete collapse of the whole structure, several damage states could be defined. The task is faced in different engineering disciplines. In practical applications intensity scales between three and ten discrete damage grades have become widely accepted. These scales are largely influenced by the way the damage is described. As [BLONG 2003] stated with citing [KALY ET AL. 1999] this can be done by a greater variety of means such as:

- Monetary loss with reference to the year in which the estimate was made
- Percentage of loss, relating the costs of damage to the construction costs
- Numerical values that are standardized on a 0-1 scale
- Macrodamage categories

Regarding the cultural, social and historical values involved, the first two methods cannot be applied. The representation in terms of numerical values obtained in the computational analysis could be used, but they are not connected to expressions of damage, such as partial or total collapse, which allow the calculation of resulting losses. Consequently, macrodamage categories have to be defined at first. The following table presents an overview of damage intensity scales after [BLONG 2003] with small modifications.

Scale name	Peril	No. of levels	Comments
Rossi–Forel	Earthquake	10	Only 7 levels refer to damage; maximum values
Modified Mercalli	Earthquake	12	Only MM6-12 relate to building damage, so effectively a 7-point scale
JMA Earthquake Intensity	Earthquake	13	Only 3 out of 8 intensities relate to building damage and only 4 out of 8 to damage of any sort
Medvedev–Sponheuer–Karnik Scale (MSK)	Earthquake	12	5 grades of damage from slight to total; 3 types of structures; definitions of quantities; maximum values
European Macroseismic Intensity Scale, (EM-98)	Earthquake	12	5 grades of damage from negligible to near total collapse; masonry and reinforced concrete structures; mean values
Beaufort Scale	Winds	7	Maximum 1-minute wind speeds
Saffir–Simpson Damage Potential Scale	Tropical cyclone	5	Based on wind speeds and surge heights; probably maximum sustained (10-minute) wind speeds
Australian Cyclone Severity Scale	Tropical cyclone	5	http://www.bom.gov.au/info/cyclone/; hazard scale with anticipated damage; based on peak 3-second wind gusts
Cyclone damage level for non-industrial buildings	Tropical cyclone	10	Damage classes for non-industrial buildings, based on Cyclone Tracy (Australia, 1974) experience; maximum values
Fujita Scale of Tornado Wind Intensity	Tornado	6	Up to 12 levels in some versions; maximum value
TORRO Tornado Intensity Scale	Tornado	11	11 points can be grouped – purports to be a magnitude scale but really an intensity scale as usually based on damage
TORRO Hailstorm Intensity scale	Hailstorm	11	http://www.torro.org.uk/hsintens.html; damage scale based on effects – hailstone size described in size categories
Hail stone diameter	Hail		Descriptive – orange, golf ball, marble etc. – converted to cm diameter; usually maximum value
Tsunami magnitude	Tsunami	7	H is maximum run-up height; intermediate values can be described and higher or lower values added; likely categories of damage attached; maximum value at individual sites
Landslide damage to buildings	Landslide	8	Separate scales for buildings and 5 types of infrastructure; and for buildings in run-out/deposition zones; maximum values
Building subsidence	Subsidence	6	Concerned only with damage to walls as a result of cracking; maximum values
Volcano Explosivity Index (VEI)	Volcanic eruption	8	Logarithmic scale based largely on volume of tephra (m^3), but also eruptive cloud height and descriptive terms
Ash Encounter Severity Index	Volcanic ash	6	Ranges from no notable damage to aircraft crash
Space weather scale for geomagnetic storms	Geomagnetic storms	5	http://sec.noaa.gov/NOAAscales/index.html; physical measure of disturbances in geomagnetic fields caused by gusts in solar wind. Indicates effects on power systems, spacecraft operations and other systems
Torino scale	Asteroids and comets	11	http://impact.arc.nasa.gov/torino/: collision probability in 21st century and kinetic energy of newly-discovered asteroids and comets. Gives general description of collision probability and likely consequences from none to global climate catastrophe
Damage state scales	All	5, 6, 7 or 10	Range of scales, providing a range of values in terms of replacement index and a central damage factor ratio
Infrastructural Stress Values	All	Continuous	Differentiates western and 3rd world disasters
Earthquake Disaster Risk Index	Earthquake	Continuous	Integrates physical, economic and infrastructural characteristics of areas at risk; maximum values

Table 5 - 1: Review of damage intensity scales [BLONG 2003]

5.3 Categorization of damage

5.3.1 General remarks

The discrete damage grades displayed in table 5-1 are often defined by *relative expressions*. [PARK ET AL. 1987] for example propose a classification based on simple observation into the five categories:

- None (localized minor cracking at worst)
- Minor (minor cracking throughout)
- Moderate (severe cracking and localized spalling)
- Severe (crushing of concrete and exposure of reinforcing bars
- Collapse

Other approaches focus on the *reparability of a building*, as it is also used within the context of Performance Based Seismic Engineering. [STONE AND TAYLOR 1993] used a four step categorization:

- Undamaged or minor damage
- Repairable
- Irrepairable
- Collapsed

Another type of measure includes *non-structural damage*, as proposed by the Earthquake Engineering Research Institute [EERI 1994]:

- None
- Slight (minor damage to non-structural element; building reopened in less than one week)
- Moderate (mainly non structural damage, little or no structural damage; building closed for up to three months, minor risk of loss of life)
- Extensive (widespread structural damage; long term closure and possibly demolition required; high risk of loss of life
- Complete (collapse or very extensive, irreparable damage; very high risk of loss of life.

A fourth and last general type of grading the structural damage is based on the *structural assessment of the structure under a second earthquake* as used by [RODRIGUEZ-GOMEZ AND CAKMAK 1990]. [WILLIAMS AND SEXSMITH 1995] state that this has the best potential of correlating with fatalities and loss of use. The four characteristics cited above were all developed for concrete structures, thus in the following two – albeit similar – proposals for masonry shall be explained.

5.3.2 Proposal used by Augusti

[AUGUSTI ET AL. 2001], who also developed an approach for the probabilistic assessment of historical structures under earthquakes, apply a seven level damage scale defined for each macroelement as follows:

- No evidence of damage
- First damages that can be detected only by an accurate examination
- Significant damages and "readability" of the mechanism leading to the collapse, but although activated it is still in its initial stage
- Clear and evident damages and "readability" of the mechanism, fully activated and in an intermediate stage of development
- Macroscopically evident damages and full development of the mechanism, with some minor part of the macroelement at the limit of collapse because of significant overall movements
- Similar to the previous one, but with significant parts of the structure at the limit of collapse and/or destruction and/or failures of other parts
- Complete collapse

Later in their numerical application they use an eleven level damage scale without further specifying the levels in order to receive a more smooth type of damage description. They assume that for a given value of peak ground acceleration the probability mass distribution of the damage to the macroelement may be described by the binominal distribution.

$$B_{(n,k,p)} = \frac{n!}{k! \cdot (n-k)!} \cdot p^k \cdot (1-p)^{(n-k)} \tag{5.1}$$

Where $B_{(n,k,p)}$ is the probability to suffer a damage of k in a scale of damage 0 to n; p is a parameter that defines the mean value for damage in a normalized damage scale ($0 < p < 1$). The parameter p is obtained by stating, that its collapse probability is equal to the probability that the level of damage is equal or larger than a threshold level n*.

$$P_j = 1 - B_{cum}(n, n^*, p_j) \tag{5.2}$$

Where B_{cum} is the CDF for $k < n^*$ of the corresponding binominal function. For the 7 level approach Augusti adopts $n^* = 4$ and for the eleven level scale he chooses $n^* = 7$.

5.3.3 The EMS approach of damage

Earthquake intensity scales explain the possible damage in detail. Intensity scales are usually used only on a regional scale with a large number of typical structures. Thus, they are inappropriate for the assessment of historical or monumental buildings. The *Modified Mercalli Intensity, MMI* for example defines scale 8 [Wood and Neumann 1931]:

> "Fright general - the alarm approaches panic. People driving motor cars can feel the motion. Trees are shaken strongly – branches and trunks will be broken off, especially palm trees. Small amounts of sand and mud will be ejected. There will be both temporary and permanent changes in the flow of springs and wells; dry wells may experience renewed flow and there may be a change in the temperature of spring and well water. Damage will be slight in structures (brick) built especially to withstand earthquakes. It will be considerable in ordinary substantial buildings and there will be partial collapse: In some cases, the shape of wooden housed will twist and contort, frame structures may throw

out panel walls and decayed piling will break off. Walls will fall. Solid stone walls will seriously crack and break. There will be some extent of landslides in wet ground and on steep slopes. Chimneys, columns, monuments, factory stacks and towers will twist and fall. Very heavy furniture will move conspicuously and overturn. “

The *European Macroseismic Scale, EMS-98* [GRÜNTHAL 1998] defines five damage grades directly referring to a building. These damage grades can be easily related to pushover curves as it was for example proposed by [LANG 2002]. So far, it was never tried to apply this scale also on vaults and other types of structural elements. Before this is done, it is helpful to understand the proposal of [LANG 2002], which will be described in the next paragraph.

Grade 1, defined as *negligible to slight damage*, is characterized by hairline cracks in very few walls and the effects on small pieces of plaster or loose parts of upper, stand-alone parts of the building. It describes a point close to the onset of the first cracks. Grade 2, *moderate damage*, is the point where many cracks occur and/or smaller partial collapse, e.g. chimneys or larger parts of plaster. Thus, this point corresponds to the part of the Force-Displacement diagram, where the nonlinear part begins. Close to this point the damage grade 3, *substantial to heavy damage*, may be found. To distinguish between the last two is not always easy. But it can be said that in damage grade 4, cracks are large and visible from far. First failures occur. In damage grade 4 described as *very heavy damage*, extensive partial collapse of walls, roofs or floors results as a consequence of the ground motions. Finally, the *total or near total collapse* is comprised in the damage grade 5. The following picture illustrates the damage grades with respect to the pushover curve of a fictitious building with at least two shear walls.

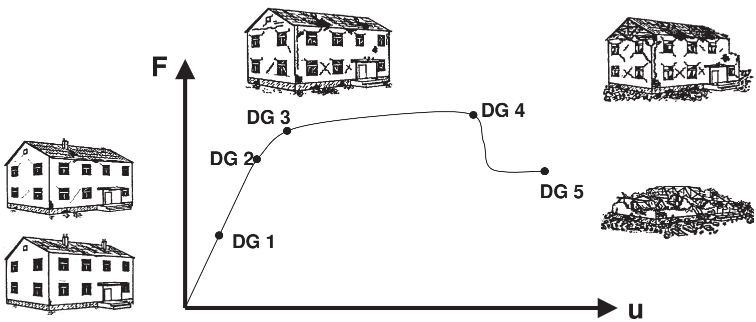


Figure 5 - 1: Connection of the EMS-98 damage to a pushover curve, after [LANG 2002]

5.3.4 Summary

The European Macroseismic Intensity Scale is the most powerful and increasingly used tool to describe both structural damage and resulting consequences. Five damage grades are explained in sufficient detail for masonry buildings, while most of the other discrete damage grades do not include a precise description of the different sizes of damage. Table 5-1 indicates that often around ten discrete damage grades are categorized. Still, damage spectra for earthquakes normally include only around five to seven different states. As figure 5-1 explains, this is enough to characterize the structural response, although damage grades 2 and damage grade 3 are somewhat smeared. Due to these considerations and because it is widely and easily applied, the EMS approach will be pursued in this work.

5.4 Existing damage indices

5.4.1 Introduction

Damage indices aim to quantify the structural damage numerically. They can be interpreted as the results of the numerical analysis and intend to relate numerical results to structural damage grades. In contrast to the discrete damage grades, damage indices are continuous. Two general groups are distinguished, local and global indices. A third type consists of insurance related indices, which are taken to be a subgroup of global indices.

5.4.2 Local damage indices

The structural damage occurring in masonry buildings may, most easily, be subdivided into two different categories. At first, damage may occur as a result of excessive deformation. Next, the damage is accumulated through repeated loading-unloading processes. Existing indices reflect this subdivision. Very early forms of *non-cumulative* indices, which are still widely used, are the *ductility* of a structure and the *interstory-drift*. The ductility is simply defined as the ratio of ultimate measure to the yield measure, where the term measure might be substituted by rotation, displacement or similar calculated results.

Nevertheless, the majority of factors fall into the class of *cumulative* parameters. They could in turn be subdivided into *deformation based; energy based* or *combined damage indices*. An excellent overview is given in [WILLIAMS AND SEXSMITH 1995]. Of all those, the most common and widely used is the *Damage Index* derived by [PARK AND ANG 1985]:

$$D = \frac{|u_{\max}|}{u_{ult}} + \beta \cdot \frac{HE}{R_y \cdot u_{ult}} \tag{5.3}$$

With u_{ult} being the maximum monotonic displacement, HE the absorbed hysteretic energy, R_y the yield stress and ß a weighing factor. HE may be expressed as:

$$HE = \sum \oint R_u \, du \tag{5.4}$$

With R_u being the stress at each point of time and u the corresponding displacement. Values for D normally fall continuously within the range from zero to one. They are connected to discrete damage states as follows:

$D < 0.1$	No damage or localized minor cracking
$0.1 < D < 0.25$	Minor damage, light cracking throughout

0.25 < D < 0.4	Moderate damage, severe cracking, localized spalling
0.4 < D < 1.0	Severe damage, crushing of concrete, reinforcement exposed
D > 1.0	Collapsed.

Most other local parameters are only variations of this basic parameter and are not explained further.

Regarding the application of local damage indices for this research, critical remarks have to be given. As it was already analysed in chapter 3, the influence of duration has a significant effect. This is particularly true for masonry structures which exhibit both stiffness and strength degradation effects. It is concluded that non-cumulative local damage indices may thus give only a rough interpretation of damage. Also, it has to be considered whether local indices should play a role at all within this thesis. Although damage is usually concentrated on some points, the number of macroelements and total areas in the structure for which the local damage indices would have to be calculated is too high to be practicable. Still, in some detailed analysis their consideration should be kept in mind. Another drawback of the local damage indices is that most of them include weighing factors [WILLIAMS AND SEXSMITH 1995], which are altering for every structure or geometrical conditions. This introduces large subjective influence in the risk assessment. Finally, most of the indices were developed for reinforced concrete failing in flexural modes. No damage index offers the possibility to relate it to possible consequences or losses.

5.4.3 Global damage indices

Global ductility is certainly the simplest engineering parameter to describe damage due to ground motions. It is defined as the maximum displacement of the uppermost floor divided by the yield displacement received by the pushover method. Other methods usually refer to averaged indices multiplied with *subjective weighting coefficients* such as the one proposed by [BRACCI ET AL. 1997]:

$$D_{storey} = \frac{\sum w_i \cdot D_i^{(b+1)}}{\sum w_i \cdot D_i^b} \qquad (5.5)$$

Where D_i is the local damage index at location i, w is a weight parameter and b is used as a parameter who emphasizes heavily damaged structural parts as b is increasing. Other parameters include the change in the modal properties.

Damage indices in insurance industry are generally economically based. In civil engineering the *damage ratio DR* is often used and determined by dividing the costs for renovation by the total cost of a building. If the damage ratio is used to predict damages for given earthquake intensities, the *mean damage ratio MDR* is used, which is defined as the best estimate or calculated mean value of the damage ratio including uncertainties in the material or a larger number of buildings, e.g. all residential buildings in a town. Other values are based on scenario calculations, such as the *Scenario Loss*. The Scenario Loss predicts damage for a precisely defined earthquake ground motions and is often used for worst case scenarios.

Global indices are highly susceptible to errors in judgement, because at first they are usually based on the summation of local indices, which already are not fully correct as described in the previous paragraph. Secondly, the introduced weighing coefficients give no objective measure, because they differ from expert to expert. Modal measures usually are able to describe locations of heavy damage and maximum damages but only insufficient knowledge is acquired about the distribution of damage.

Comments made on the previous pages lead to the conclusion that the existing indices cannot be used for the earthquake risk assessment of historical structures. Therefore, a new approach must include the particularities of masonry and should finally enable the derivation of potential consequences as a result of structural damages. The following text will deal with this task with reference given to the results obtained in the structural analysis of chapters 3 and 4.

5.5 Correlating analysis output and damage

5.5.1 Analysis output

As it was described in chapter 5.3.3 the EMS approach can be easily applied to the numerical results obtained by the pushover method for buildings consisting of a given number of shear walls. Still, significant drawbacks have been explained depending on the pushover method at first and additionally stating that vaults, domes or arches cannot be addressed by this approach.

To identify which output parameter of the nonlinear dynamic simulations of chapters 3 and 4 is the best descriptor of damage, the results of the previous calculations may in a first step be correlated to the top displacement. The pushover method uses the top displacement as the main indicator of structural damage. Consequently, tests will in a first step correlate the output parameters to the top displacement. Afterwards, better indicators will be derived. A first plot of correlations, using the results of chapter 3 is presented in the next figure.

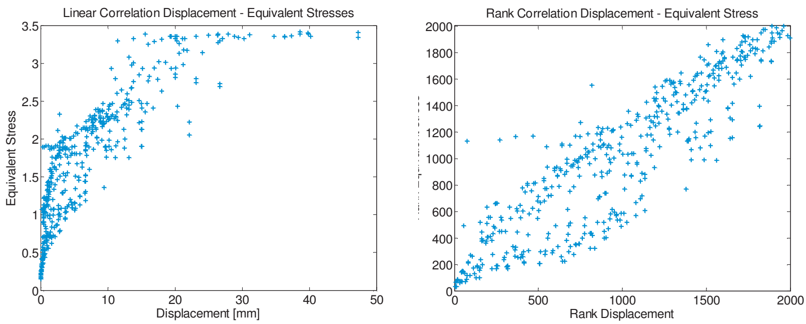


Figure 5 - 2: Correlation of displacement and equivalent stress

Equivalent stresses, as shown in figure 5-2, exhibit the highest correlations of all stresses, especially for higher excitations. Shear and tensile stresses are not correlated as well as it is depicted in figure 5-3. The correlations would be more evident if plotted by their rank order, but the dispersion is already clear enough if plotted on a linear scale.

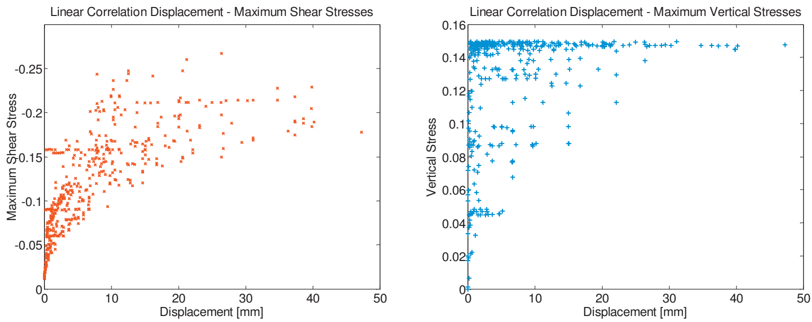


Figure 5 - 3: Correlations for shear and tensile stresses

The strains do not show a sufficient correlation either; especially global plastic strains accumulated over the time are not well correlated with the total displacement as figure 5-4 indicates.

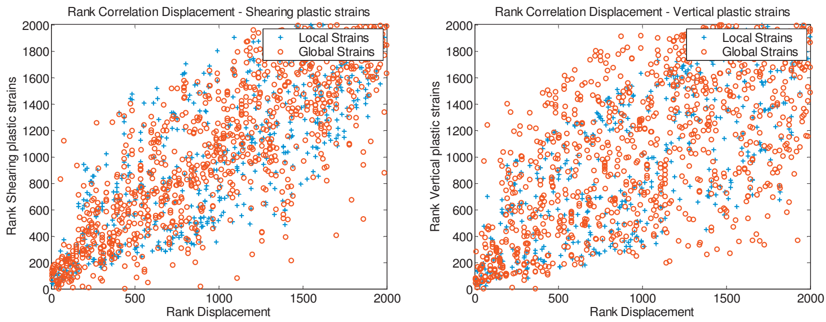


Figure 5 - 4: Rank correlation of shearing and vertical strains

Finally, the two internal damage variables are the correlated best with the displacement. The nonlinear dependence between mortar damage and displacement may be seen in figure 5-5. Similar plots would be obtained for the brick damage. Figure 5-6 plots brick and mortar damage versus the displacement in rank order correlations.

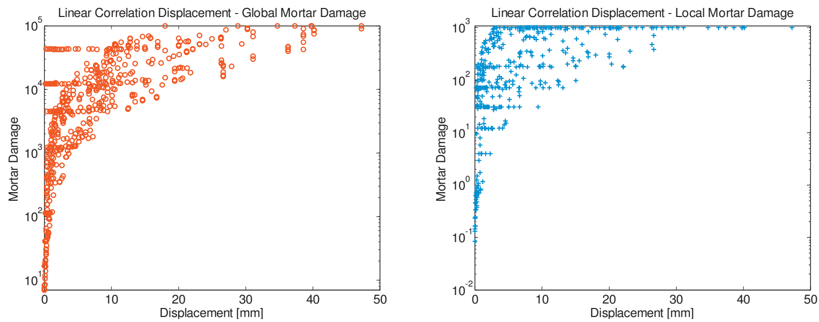


Figure 5 - 5: Linear correlation of mortar damage versus displacement

The upper and lower figures also point out that for high damages, the global mortar damage is less scattered and provides a better correlation as compared to the local damage. This is important for the application and latter definition of damage grades, which will be explained in chapter 5.5.3 and the application in chapter 7.

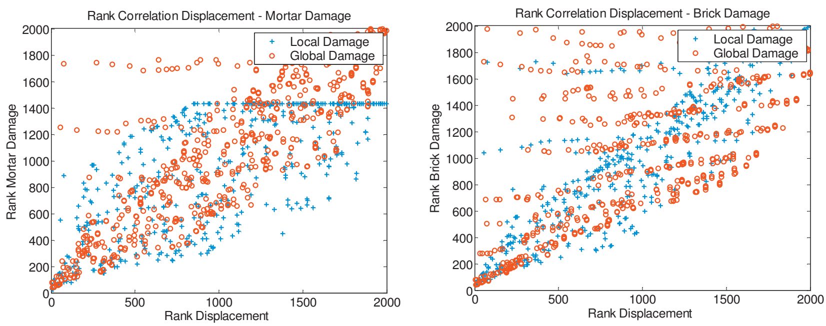


Figure 5 - 6: Rank order correlation of mortar and brick damage with the displacement

In the upper figure some outliers for higher damage are shown. These occurred as the consequences of earthquakes, whose significant duration was above 20 seconds. These earthquakes were included to assess the importance of duration. This picture underlines that even for a small displacement high damage as a consequence of long durations might be suffered. The results indicate that the damage parameters are better suited as descriptors of damage than the top displacement.

5.5.2 Discussion on the use of the parameters

Certainly the use of damage parameters cannot be justified merely by relating them to the top displacement. This would be false from the beginning, since top displacement is used as a damage measure in pushover analysis, which is different from the time-history approach used. High localized, but even higher global damage might be suffered as the consequence of longer but not very extreme ground motions, such as the results for the long earthquakes illustrated in figure 5-6.

The detailed analyses of some selected results of the database revealed that the best indicator for the loss of strength and stiffness of the structure is the global mortar damage, while local effects are best described by the local brick or local mortar damage. Remembering that these parameters – if exceeding a value of one – indicate that a crack occurs in the element, the two internal damage variables may also be seen as values able to describe visual structural damage. Both variables are evolutionary and able to describe the accumulated damage in the structure. Although the absolute values of the damage variables may reach extremely high numbers, the major loss is described by values in the range of zero to ten for common softening values. This effect is once more described in figure 5-7, presenting also a close-up for smaller numbers.

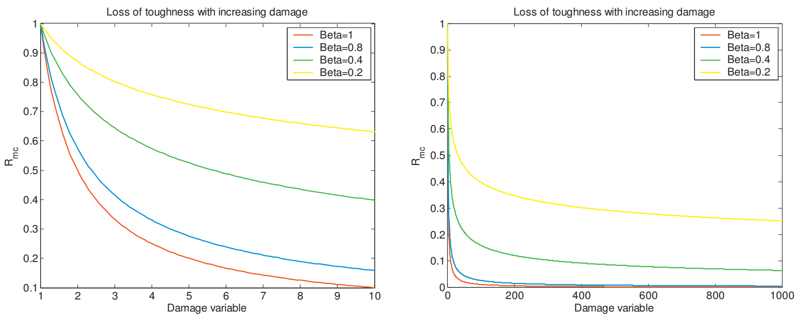


Figure 5 - 7: Loss of toughness in dependence of the damage variable

With all this information given, the numerical results are now correlated to the damage grades of the EMS-98. The special usefulness of the damage parameters will even more obvious after the connection has been established.

5.5.3 Proposal for the definition of damage degrees

The first visible cracks occur if the damage parameters exceed a value of one. It is not important, whether the brick or mortar damage variable is considered. It can thus be concluded, that damage grade 1, which is defined as *negligible to slight damage* and characterized by hairline cracks in very few walls occurs when either one of the two internal damage parameters exceeds the threshold value of 1. Also, for the purpose of this study, it is sufficient if the threshold is exceeded in a small part of the model. In sample applications, four finite elements proved to be sufficient. Grade 2, *moderate damage*, is characterized by many cracks and/or smaller partial collapse. For walls, this point corresponds to the beginning of the nonlinear part in the pushover diagram. Generally, since the term “many” is used by the authors of the EMS-98, the damage must be suffered in more than one wall corresponding to more than one location. Alternatively, if only one part of the structure is considered, e.g. a single wall or a single vault, than a more severe damage has to be suffered by the structure.

Close to this point the damage grade 3, *substantial to heavy damage*, may be found. First failures occur for small and slender parts of the structure which are described as chimneys in the EMS-98. This damage grade also describes heavy non-structural damage. To characterize this damage grade, it is proposed that the damage variable should exceed values of 10 in more than 10 percent of the wall width in case of walls or at least in two separate places, if vaults or arches are considered.

The first three damage grades resemble each other closely. While the first is of numerical importance, it is practically not of any use for the seismic risk assessment, since it is without major effects on the appearance and structural reaction. Damage grade 2 and damage grade 3 on the other hand describe the onset of the nonlinear phase in the pushover diagram and the start of cracking in the masonry in varying seriousness. The differentiation between the two damage grades is enabled easily with respect to the material parameters.

Damage grade 4 and damage grade 5 are complicated to distinguish as well. In contrast to the first three grades, grade 4 and grade 5 are close to the point of collapse. As [LANG 2002] proposes, both correspond to the point of collapse. Damage grade 4 corresponds to the collapse of single walls and grade 5 to the collapse of the whole building. If dealing with macroelements, which usually consist only of one wall, both grades would be the same. This is seen as a great disadvantage for the analysis of single elements. Especially, because the EMS-98 describes damage grade 4 as very heavy damage, which is not equal to structural collapse. The approach proposed in this work will hence differ between grades 4 and 5 for one single macroelement. For the numerical results, damage grade 5 may be easily interpreted as numerical failure. The author is aware that numerical collapse and structural collapse are not identical. Since damage grade 5 is clearly described as total or near total collapse this is nevertheless sufficient. It is thus more complex to evaluate significant features characterizing damage grade 4.

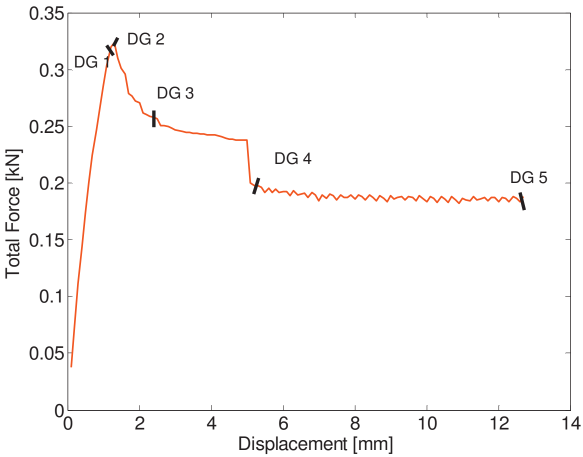


Figure 5 - 8: Force-Displacement relationship for a steadily increased top displacement

To find a definition for DG 4, it has at first to be related to DG 3. The most important distinction between the two damage grades is that while DG 3 is located close to the first decrease in strength and stiffness, grade 4 is found close to the point of collapse, which corresponds to a point close to the ultimate strength and stiffness reduction. Figure 5-8 shows a Force-Displacement diagram which was gathered by increasing the top displacement of the structure. The proposed damage grades are also assigned to the curve in this figure. It can be seen clearly that at damage grade 4 the structure is close to failure and only capable of transferring further loads by frictional mechanisms, which are active because a high load was applied at the top of the wall. The structure is transferring the loads by frictional mechanisms, although it would already have failed without any load applied to the top. If this happens, extremely high values for the damage variables are obtained.

Results from chapter 4 show that for some calculations high values of these parameters are achieved, although the structure does not fail. The following figure repeats the results presented in chapter 4. As it can be seen, values of damage between 500 and 1000 are seldom achieved, although the value of 1000 is reached several times.

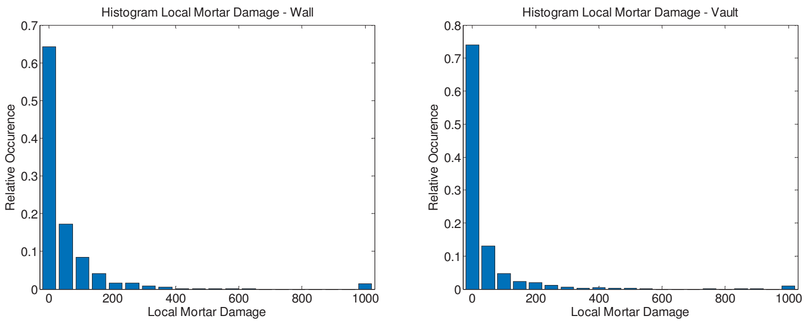


Figure 5 - 9: Histogram of damage variables

Summarizing the observations, damage grade 4 is defined to occur, if damage variables exceed values of 1000 locally in four neighbouring elements or alternatively if the damage variable exceed values of 100 in larger areas of the structure. The term “large” is defined as more than 1/3 of the width of a wall or at least two parts with more than 4 elements in the structure. Some examples, based on numerical simulations are given in the subsequent figures, starting with the wall, that was analysed in figure 5-8.

The criteria for the characterization of the damage grade with the applied material model are summarized in table 5-2.

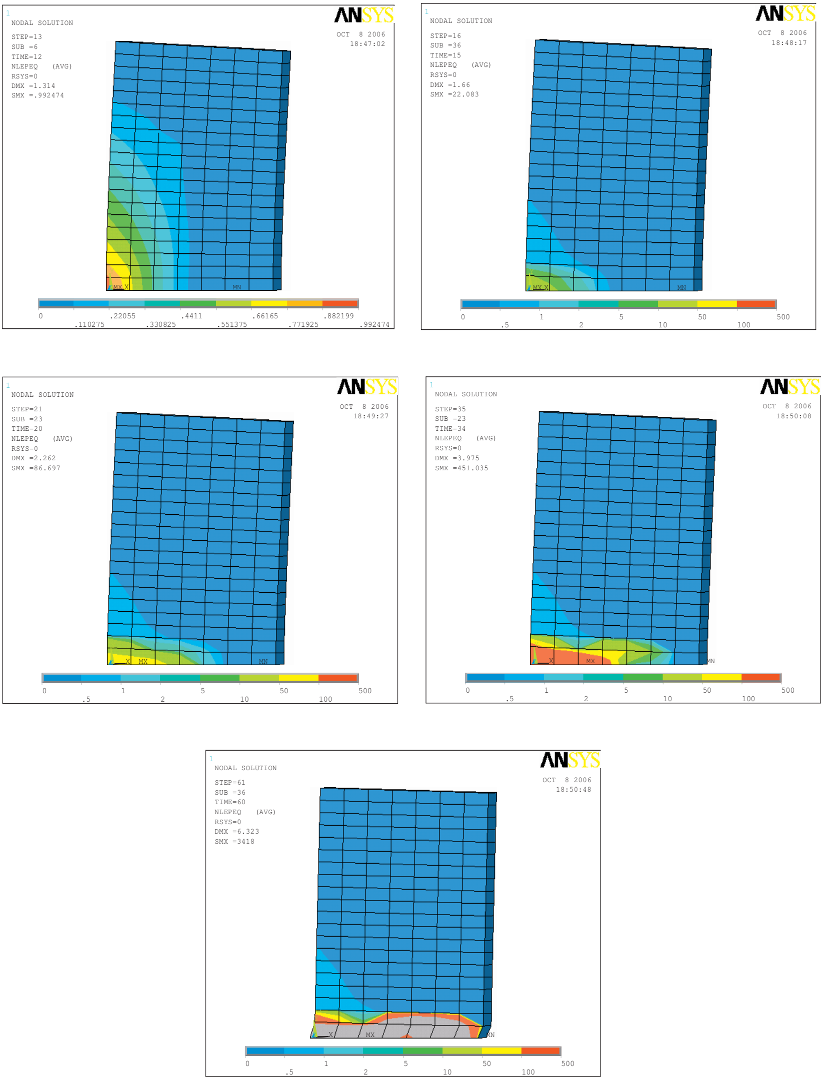


Figure 5 - 10: Examples for damage grades in a wall, DG1 upper left, DG2 upper right, DG3 middle left, DG4 middle right, DG5 lowest

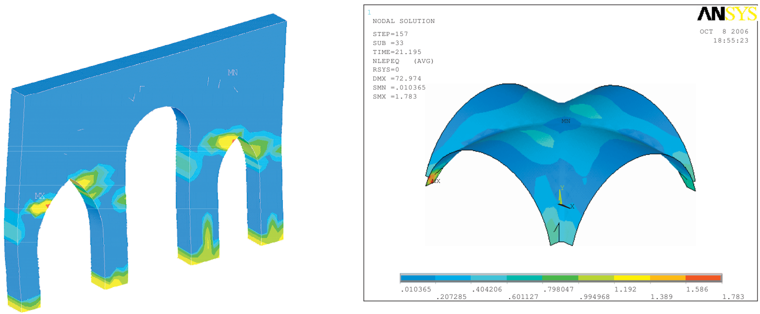


Figure 5 - 11: DG3 for an arch, yellow areas indicate mortar damages higher than 10 and red areas that the damage is higher than 100 (left), DG2 for a vault – twice a value above 1 was reached.

Damage grade	Criteria
1	Either brick or mortar damage variable exceed the value of 1 in at least 4 elements or the value of 5 in one element at one location
2	Criteria one is occurring in more than one location, damage variables range 2 - 10
3	Either brick or mortar damage variable exceed the value of 100 in more than one location or at least 10% of the wall width or in more than one location is over the value of 10
4	Either brick or mortar damage variable exceed the value of 1000 in at least 4 elements or exceed a value of 100 in more than 30% of the wall width or more than one location
5	Numerical collapse

Table 5 - 2: Damage grade classification

5.6 Determining the probability of damage

The probability of damage may now simply be determined in accordance to the results of chapter 4. The local mortar damage is a very good indicator for the results of the single walls and DG 1 to DG 3. Global damage provides supplementary indicators for DG 3 to DG 5. No damage occurs, if both internal damage variables are less than one. If any value falls between one and two, damage grade one is achieved. Local damage resulting in values between two and ten is classified as damage grade two. A more detailed view is not necessary, for they both do not have any effects on the possible consequences of structrual damage. Values between ten and 100 are categorized as damage grade 3 and all local values above the threshold of 100 are considered to be damage grade 4. Numerical failure is taken to be identical to DG 5. Numerical failure might occur as a consequence of convergence problems in the model, which is not the same as real structural collapse. Still, DG 5 is described in EMS-98 as total or near total collapse, so that the definition derived is justified. In this way, it is possible to calculate the probability of each damage grade for a given ground motion. To do so, the histogram obtained in chapter 4 must be divided according to the data presented in table 5-2.

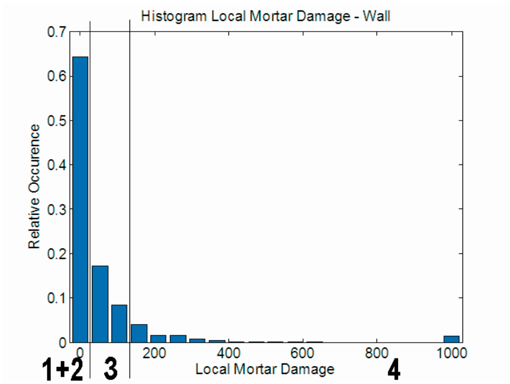


Figure 5 - 12: Classification of damage grades for the histogram obtained in chapter 4

The application shown in chapter 7 fully describes the approach and the results based on a sample study. Before this task can be comprehensively performed, it is necessary to conclude the work of this thesis by explaining the analysis of possible consequences, relating the final results of the risk assessment procedure to the risk measures explained in chapter 2. This will be done in chapter 6.

Chapter 6

Loss assessment and risk comparison

6.1 Introduction

In the last chapters, the foundation for the risk analysis was laid. It is now possible to express the vulnerability of historical structures in truly probabilistic terms. What has been done so far was only the analysis of the structural behaviour. Still missing is the evaluation of possible consequences of structural damage. Although a survey of the structural damage suffered by churches during the Umbria-Marche earthquake was performed by [LAGOMARSINO ET AL. 2002], an extensive literature review has not lead to any information about what types of values might be lost or affected if churches are subjected to earthquake loads. For this reason, a database was set up for two main purposes. At first, main factors to describe possible consequences had to be identified. Special importance was devoted to possibly affected human lives. As a second aspect, sufficient data had to be collected in order to be able to compare different churches. This was necessary because – as will be seen later – it is difficult to transfer cultural and social values into countable numbers. Moreover, a larger database offers the possibility to compare structures and to discover those with a comparatively high risk. In this way – by comparison – it will be possible to identify those structures with a high risk and those with a low one, without grading the intangible values themselves.

6.2 Specification of the data assignment

In order to start the research and create the database, at first some ideas had to be developed about what kind of data is important and how it is best collected. Those first concepts determine the design of the survey instrument and the methods to obtain the desired information.

The principal task is to be able to include the most important contributions into the assessment of the overall losses. Certainly, losses are not only restricted to losses of human life or the economically determined reconstruction costs. Neither do they simply consist of a combination of both, nor is it possible to describe them in terms of such parameters as the Societal Value of a Statistical Life. Instead, as described in chapter 2.1.3, all different types of possible losses have to be determined. Therefore, the four main – be it direct or indirect – consequences as plotted in figure 6-1 have to be considered.

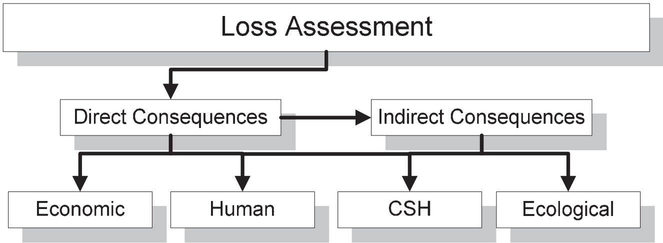


Figure 6 - 1: The four main types of possible losses/consequences

In this work, ecological consequences do not contribute to the risk. Therefore, emphasis is laid on the three other components of economic, human and CSH (i.e. cultural, social and historical) consequences. The CSH consequences might be seen as a melting pot of diverse kinds of risks. Whereas one might argue – and certainly justified – that some risks, e.g. political ones, are not considered, it is understood that those are already covered by the social risks included in the CSH values by definition.

The primary objective of including CSH values in the risk assessment is accompanied by a number of subsidiary goals, which – although they do not directly affect possible losses – are still enhancing the results of this study. The complete framework of objectives is listed in the following.

• Derivation of probability curves for possibly affected human lives
• First estimations of reconstruction costs
• Measures to evaluate each component of the CSH-consequences
• Generate data about typical structural conditions of historical churches and the area they are located
• Collection of geometrical data
• Yearly costs for reconstruction and renovation to relate them to formula 2-17

Table 6 - 1: Subsidiary goals of the study

6.3 Determination of research tool

6.3.1 Data collection

At first, it was necessary to identify historical churches, at best in earthquake prone areas. Official institutions, such as dioceses and authorities for monumental conservations were not able to provide a list of historical structures for diverse reasons, mostly because these lists do not exist or are spread over different departments. Gratefully, most dioceses provide a link on their website to most of the churches, which belong to their regional supervision. In turn, those websites often provide information about the construction year and additional data about the building. This enables a web search, which was used as a first step of data collection. While in Germany single churches had to be accessed via the website of the diocese, [CHIESA CATTOLICA 2005] could be used as a similar source in Italy.

By this method, a first database was established and data about the number of people belonging to the religious community and the age of the churches was listed. Additionally, German dioceses provided information about the size of the religious communities belonging to each church and the number of visitors, which is commonly counted twice a year. This information offered additional important background data. Nevertheless, a more detailed survey was necessary, which enabled a precise categorization of churches according to possible losses.

6.3.2 Survey Instrument

For this reason, it was necessary from the beginning to contact the churches directly. To cover a wide range of structures with feasible and justifiable effort, a questionnaire was developed to meet the goals defined in table 6-1. The design had to be such that even if the person asked is not accustomed to the task, he or she should be able to answer the questions. Consequently, the layout had to be designed general enough to be quickly understood. At the same time, it was necessary to generate precise information to classify structures according to the inherent risks. The final questionnaire is attached in appendix H and consisted of 7 parts and a total of 33 questions. One part aimed at collecting general data; while the other six were designed according to meet the tasks of table 6-1. It was planned in such a way that most questions required only crossing the correct answers. Only few questions asked for a short written answer.

Although a questionnaire in engineering topics might be regarded as a rather simple task of surveying, various publications explain that the design and layout of such a questionnaire is a critical and important task indeed. Hints on the design may be found in [PLAPP 2003] or [KUTSCHKE 2002]. Some of the details included are listed in the following table.

Detail	**Description**
Cover letter	Official head, exact date, personal address, explanation of purpose and usefulness of the study, importance of each recipient, declaration of anonymous data, phone and email address for questions, encouragement for participation, thanks for participation, personal signature
Package	White letter, prepaid letter for response
Dispatch	In the middle of the week, regarding holidays
Pre-Study	Check the usefulness and anticipation of recipients in a first test study

Table 6 - 2: Details on the shipment and design of questionnaire and cover letter

Within a first step, all Italian churches falling into the class of monuments, protected by the Italian ministry of cultural goods and activities and listed at [BENICULTURALI 2006] were used for a first sample study to test the usefulness of the questionnaire. The response to the questionnaire was surprisingly high, i.e. a response ratio of approximately 40% was achieved, and revealed that only slight adjustments had to be incorporated to further improve the questionnaire.

Originally, it was planned to assess the economical importance by the number of shops in the surrounding area; including the number of visitors and the employees, if known. In 13 out of 15 responses these questions were either not answered or not wanted to be answered, as it was expressed in one comment, revealing that gathering this kind of information would be too ambitious for this study. In order to keep people motivated, these questions were not included in the revised questionnaire. Also some small adjustments were made concerning the information about the cultural significance with the purpose to animate more persons to send back the information. This was done by providing a detailed list of answers to avoid the great variety of answers, which were obtained in the test.

In the second step, the revised questionnaire, which now consisted of only 30 questions, was sent to additional 140 churches in Italy by mail. To gather further information, the questionnaire was designed as a form that could be filled out electronically. Then, this form was sent by mail to 90 churches in Italy. While the response to the mail was still well, the response to the email letters was poor, reflected by the return ratio of 2.2%. The information was thus collected only by proper mail in Germany.

6.3.3 Regional location

The purpose of the study required the churches to be located in areas where they are at least exposed to a low seismic hazard. While in Italy it was considered to be sufficient, if the church was located south of the river Po, in Germany only churches situated in one of the three areas with a significant seismic hazard were included. The specifications given in [DIN 4149] helped to determine the suitable locations. General data collection included churches of the dioceses listed in table 6-3. The detailed questionnaire survey was only performed for churches within the lower Rhine embayment, i.e. of the diocese Cologne and Aachen.

Location	**Diocese**
Lower Rhine Embayment	Cologne (only locations with seismic hazard)
	Aachen
Upper Rhine Valley and Hohenzollern Drift	Freiburg
	Rottenburg-Stuttgart (only locations with seismic hazard)
Thuringia	Erfurt (only locations with seismic hazard)
	Dresden (only locations with seismic hazard)
	Magdeburg (only locations with seismic hazard)

Table 6 - 3: Dioceses and regional location of the German churches

6.3.4 Components of the survey

As already mentioned, the survey consisted of seven parts. It was accompanied by a cover letter to introduce the topic, to explain the subject and to ask for filling out the form, even if not all questions could be answered. In the beginning some simple questions were placed to ease the introduction for the respondent and to encourage readers to fill out the full questionnaire. Collected data classified the regional characterization of the church and some very general structural characteristics. This was done to evaluate the representativeness of the sample and to determine special features of the churches included in the study.

It was a sophisticated task to assess the value of the cultural goods inherent in churches. To keep the effort for complicated the questionnaire simple, only four questions were included in the study. Next to the collection of data concerning the number and the age of movable or non-movable cultural goods, it was asked for the subjective impression, whether the church is known for its cultural goods and within what region. Also, information concerning the monumental protection was collected. Within the first test of Italian monuments, it became already evident that this data results in a great variety of answers which do not provide comparable information. As a result, the evaluation of the tourists visiting the church moved into the centre of focus.

Another major task was to be able to assess the possible number of lives lost. Therefore, the evaluation of the number of persons in the church was emphasized. Persons were grouped into three main categories. At first, data were collected about the number of tourists visiting the church per year; including the distance they have travelled to visit it. This is based on the *travel cost approach* [ICOMOS 1993] used in *Business or Heritage Economics*. The travel cost method is used to assess the importance of cultural sites. Secondly, data was gathered concerning the persons living in the church. In a third step, the amount of visitors of religious events was estimated by a total amount of four questions related to this matter. The information on human risk was rounded off by two questions dealing with the total of size of the religious community.

It was also planned to offer details on the renovation and reconstruction costs as well as information on the economical influence the church has on surrounding museum, shops or bars. Respondents proved to be unwilling or unable to offer information about these questions. Consequently, only three basic questions were incorporated in the final version of the questionnaire. They collected information about the costs spend for renovation and reconstruction with the idea to relate those costs to formula 2-17 [PROSKE 2004].

At the time the questionnaire and the general research topic have been set up, the question arised, whether the majority of churches are in general in an insufficient structural condition. This is why three questions asking for structural deficiencies were also included in the study. Another hypothesis was that the geometrical size of the church might be related to its cultural value and total amount of human risk. Therefore, the final questions covered the structural layout and the single components of each church. Eventually participants were asked, if they are interested in the results of the study and to give any comments about the questionnaire itself.

6.4 Remarks on the data pool

6.4.1 Response analysis

Expected return ratios of questionnaires are usually quite low and normally fall in the range of 10 to 20 percent [PLAPP 2003], [KUTSCHKE 2002], [GRAFSTAT 2002]. With this data in mind, the response must be graded as exceptionally good, despite the rather long questionnaire consisting of 8 pages. In the authors opinion this is not only due to the fact that the questionnaire was designed well and easy, but also as a consequence of the highly motivated people receiving and answering the questionnaire. This statement is underlined by the fact that four questionnaires were sent back after the proposed date of return with longer explanations and sincere excuses. Although this is not of direct importance for this thesis, it is interesting to note that this research obviously addressed a task, which is truly important for most persons responsible or working in a historical church. In detail, the response ratios were as follows:

Italy

- Of 40 Beniculturali questionnaire 15 returned, corresponding to a ratio of 37,5 %
- Of 140 additional questionnaires 30 returned, corresponding to a ratio of 21,4 %
- Of 90 questionnaires sent by email 2 returned, corresponding to a ratio of 2,2 %

Germany

- Of 100 questionnaires 45 returned, corresponding to a ratio of 45 %

If the free space for comments at the end of the questionnaire was used, additional information about the churches was offered in all 23 cases, when a comment was written down. No criticism was given after the first sample test.

6.4.2 Representativeness of the sample

If the response of the questionnaire is used to draw general conclusions, the sample must be representative of the parent population. The representativeness of the sample is analysed for three different characteristics because these data were evaluated for a larger sample of churches. The three classes include:

- Size of the religious community
- Age of the churches
- Number of visitors attending the religious events

Results explained in chapter 6.5.1, presented together with the evaluation of the parent population, reveal that the sample reflects the basic population well in all three categories.

6.5 Evaluation of the collected data

6.5.1 Observations about the parent population

The three categories mentioned directly above were analysed for a larger number of churches. At first, the size of religious communities in Italy and Germany was assessed. Therefore, information about 1456 churches given in [CHIESA CATTOLICA 2005] was analysed. To include regional differences, all available data of the diocese of Abruzzo-Molise was used.

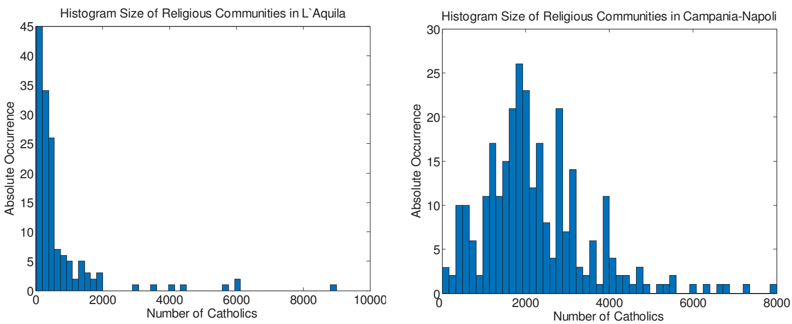


Figure 6 - 2: Size of religious communities in rural areas (left) and municipal areas (right)

Additionally, data for the municipal area of Campania-Napoli and the rural Piemont-Aosta were included. The size of the religious communities may vary largely depending on the regional situation. Figure 6-2 explains some typical characteristics. Rural areas are dominated by a number of smaller communities whereas for communities located in cities, the mean number of the population is higher. Also the histograms exhibit a different shape. Although this difference exists, if local characteristics are analysed; on a national level the distributions are not varying that much as figure 6-3 reveals.

Data about the size of parochial communities in Germany was including data provided from the diocese of Aachen, representing a total amount of 537 churches; the result is also plotted in figure 6-3. One might argue that other dioceses would exhibit different distributions and means. However, it is assumed that all German dioceses might be described by the same characteristics, because, if compared to 1500 analysed communities in Italy like it is shown in figure 6-4, a similar distribution is obtained. Thus, on a national scale, which will be important for the assessment of the earthquake risk, similar distributions may be considered.

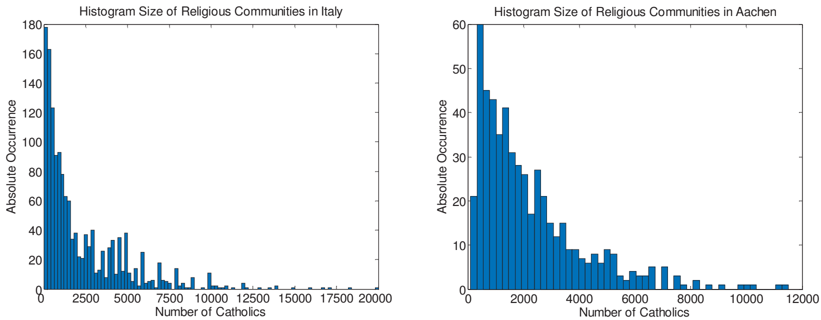


Figure 6 - 3: Size of religious communities in Italy (left) and Aachen, Germany (right)

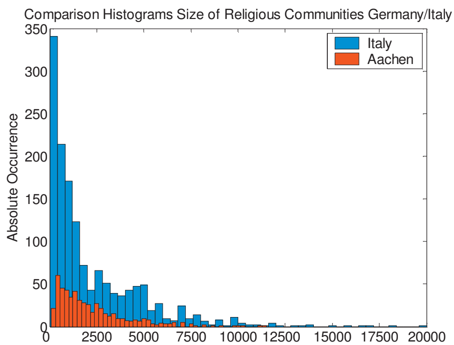


Figure 6 - 4: Comparison of religious communities in Italy and Aachen, Germany

To clarify, whether the sample is representative for these distributions, the results of the questionnaire are compared to the distributions obtained above. This is done in the following table. It can be seen that the values are close enough to state that the sample is sufficiently representative. No differences between German and Italian communities could be observed.

Size of Community	Percentage of churches in the questionnaire including Italian and German churches	Percentage of churches in the diocese including Italian and German churches
< 30	2 %	1%
30-50	2 %	1 %
50-100	9 %	4 %
100-300	11 %	12 %
300-500	14 %	10 %
500-1000	10 %	16 %
1000-3000	20 %	29 %
>3000	32 %	27 %

Table 6 - 4: Representativeness of the questionnaire sample regarding size of religious community

For the Aachen district, data about the mean number of visitors during a mass and the percentage of each religious community attending the masses was available. This information is collected on the Sunday before Easter and the second Sunday in November every two years. Visitors are counted for every mass. The plots below show the total number of visitors on both Sundays divided by the number of masses offered and the resulting percentage of the religious community.

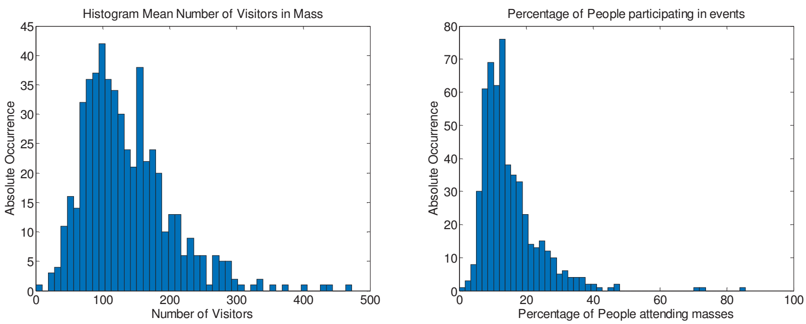


Figure 6 - 5: Number and percentage of persons attending religious events

Originally, it was planned to estimate the number of persons attending masses based on the number of persons belonging to the community. But, as the figure 6-5 shows, the percentage of people attending masses varies largely and cannot be assumed on the base of the size of persons or area belonging to the religious community. As plotted in figure 6-6, a slight trend is visible that within larger communities a lower percentage attends the religious events. Nevertheless, the scatter is still too large to draw precise conclusions.

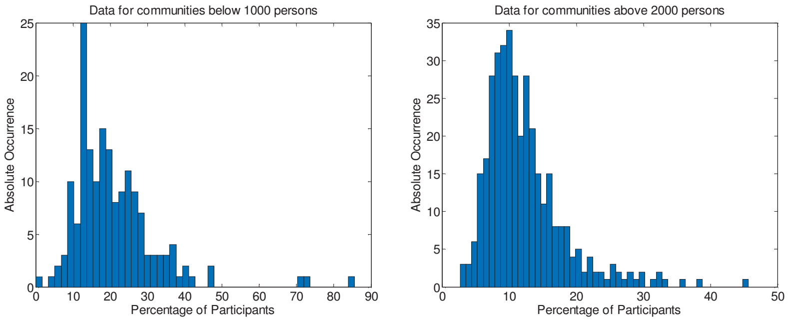


Figure 6 - 6: Comparison of participation for small (left) and large (right) religious communities

A larger data collection was also performed respecting the construction year of the churches. This task was only performed for German churches. According to the regional background given in table 6-3, the construction year for a total of 643 churches was determined. This is the total amount of churches built before 1960 in earthquake prone areas in Germany, which offered sufficient information about the date of construction on their website. The term construction year is somewhat difficult, because several additions and alterations are usually added. For this study, tower and nave were seen as individual parts and the older one was determining the construction year of the church. If the church was destroyed, e.g. in the war, and reconstructed afterwards it was not included in this study. The results are plotted on the left in figure 6-7.

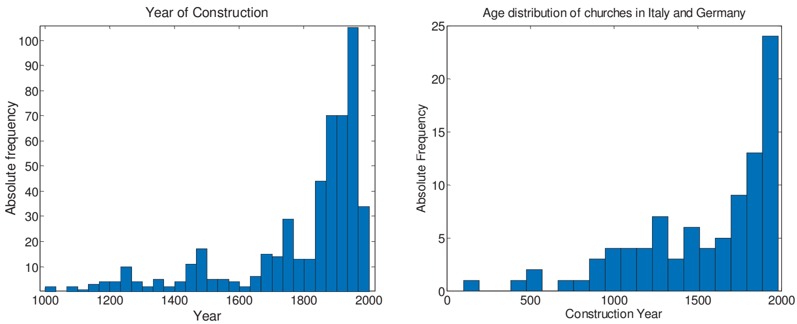


Figure 6 - 7: Construction year of German churches (left) and the of the churches analysed in the questionnaire (right)

The previous figure also compares the results of the questionnaire given on the right side to the result of the larger study. Although the general shape is similar, it can be seen that the questionnaire included churches built before the year 1000. This is mainly a contribution of Italian churches. The subsequent plot describes the results of the questionnaire in a more detailed manner, by subdividing the results into the German and the Italian part.

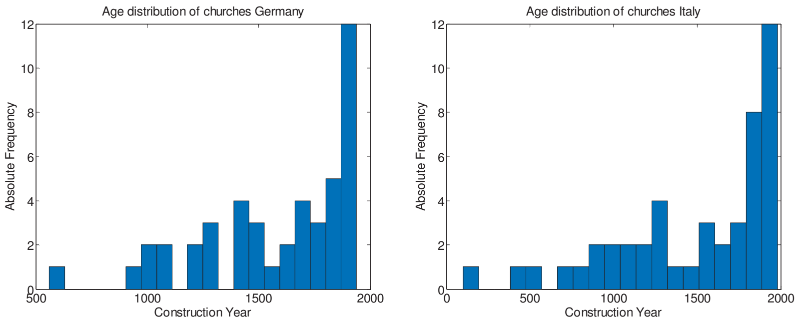


Figure 6 - 8: Construction year of churches in Germany and Italy

The mean age of the distributions is given in table 6-5. Although Italian churches – due to the contribution of the Beniculturali monuments – and the German churches of the questionnaire are older than the mean of the total amount of churches analysed in Germany, the sample is still assumed to be representative enough. This is because no parameter, especially not the number of tourists, could be correlated to the age of the structure. Evidently, an old church does not necessarily result in a higher number of visitors. The type of distribution is similar, so that the full range of the parent population was covered in this study. Concerning the assessment of the historical risks, remarks about the influence of the mean construction year will be given in chapter 6.6.2.

Data Source	Mean Construction Year
643 German churches	1795 for construction before 1960 1690 for construction before 1945
45 German churches, questionnaire	1572
47 Italian churches, questionnaire	1486

Table 6 - 5: Mean construction year of churches

6.5.2 Results of questionnaire analysis

6.5.2.1 General characteristics

This chapter will explain the responses to each question of the questionnaire. Concerning the general situation of the churches, it can be concluded that most of them are in very close vicinity of additional buildings:

- 35.87% are directly adjacent to other buildings
- 42.39% are less than 10 metres apart from the next building
- All others, i.e. approximately 22% are more than 10 metres away

The type of adjacent buildings was not specified further. Larger complexes often include museums, stores or only bureaus. What remains is the statement that about 78% of all churches are in close vicinity to neighbouring buildings, so that they might damage them in case the church collapses during an earthquake event. This is also underlined by the responses to question 3, whose result is given in table 6-6. According to the answers, most churches are situated in populated areas; rather few are located in such a way that they may be defined as single buildings.

Single building	17.36%
Villages	34.78%
City	31.52%
Metropolitan area	16.34%

Table 6 - 6: Results of question three

Not only the vicinity to other buildings contributes to the increase of risks, but also the fact that local conditions raise the exposure towards earthquake risks. What is most important is that 52 out of 92 responded that the church is located close to a supporting wall or a smaller cliff, which corresponds to 56.5% of all the answers. Ten more, roughly 11 %, stated that the church is located closely to a smaller or bigger river, with close meaning less than 10 metres. With respect to the soil-structure interaction and the complex geological impact on the response of the structure, this is certainly a fact not to be disregarded.

The date of construction was already evaluated in the previous chapter. What can be added is that roundabout 84% of all churches experienced larger constructions works after the first erection, in most cases an abside or maintenance buildings were added. The architectural style the structure can be assigned to, is given in the table below. It is also reflecting the construction age of the buildings.

Style	Germany	Italy	Total
Pre-Romanesque	4.44%	10.64%	7.61%
Romanesque	28.89%	19.15%	23.91%
Gothic	37.78%	10.64%	23.91%
Renaissance	0.0%	6.38%	3.3%
Baroque	6.67%	8.51%	7.6%
Other	22.22%	44.68%	33.69%

Table 6 - 7: Architectural style

It can be seen that in Germany Romanesque and Gothic churches dominate, while in Italy the styles are more evenly distributed. The high amount of Gothic churches is also visualized in the small peak in the age histogram shown at the time of 1500 in figure 6-8. The definition “Other” covers modern churches and those cases, where the building could not be clearly assigned to one architectural period.

6.5.2.2 Evaluation of the cultural goods

The responses to the questions dealing with the number and age of cultural goods within the church scatter largely. Also, the results could in no way be correlated either with the recognition of the church or the number of visitors. Thus, the results were neglected for the further study.

Nevertheless, the results suggest that the church as a structure itself is the major reason for a visit for most persons. The number of cultural goods inside the church does not seem to influence the behaviour of tourists. While this might not be true for churches of pilgrimage, such as St. Marcus in Venice, Italy or the cathedrals in Aachen or Cologne, Germany, the result is true for the majority of churches. Since the data did not prove to be useful, two more attempts were included to see, if the cultural value might be measured on the basis of simple information. Results showed that most of the analysed churches fall under monumental protection, which is true for 87 % of all churches, 7 responses, i.e. 7.5%, do not fall under any regulations. Five questionnaires, i.e. 5.5% did not reply to this question. Useful additional information on restrictions and reasons for not responding was not given. The reputation of the church was assessed by the respondents as follows.

Reputation	Germany	Italy	Total	Value for figure 6-9
Village	4.44%	8.51%	6.52%	10
City	31.11%	46.81%	39.13%	20
Region	53.33%	12.77%	32.61%	30
Country	0.0%	4.25%	2.17%	40
Adjacent countries	6.67%	17.02%	11.96%	50
Worldwide	4.44%	6.38%	5.43%	60

Table 6 - 8: Subjective assessment of the reputation of the church

Despite the contribution and effects of structures which belong to the Italian monuments under the Beniculturali supervision, it is nonetheless obvious that results are influenced by subjective perception and slightly exaggerated. As will be seen in the next paragraph, the results show no correlation to the more important number of tourists.

6.5.2.3 Human risks

The results of the evaluation of the cultural goods were not very useful apart from some minor aspects. This was already expected, after the results of the first test when the Beniculturali monuments were examined. For this reason, the first two questions dealing with human risks also involved valuable information about the importance of the structure for tourists. At first, the number of tourists visiting the place was analysed. Although only rough numbers were given and in most cases the tourists were not counted but only estimated, the response clearly showed more reliable results.

The high percentage of more than 10 000 tourists per year is again a result dominated by the 15 Beniculturali monuments. The results are also well correlated with accumulated data on guesses were the tourists come from. The distance the tourists travelled was estimated in four steps listed in table 6-10.

Yearly number of tourists	Percentage of Response	Value for Figure 6-9
Less than 10	23.91	10
10-50	8.70	20
50-100	14.13	30
100-500	14.13	40
500-1000	11.96	50
1000-5000	11.96	60
5000-10000	4.35	70
More than 10000	10.87	80

Table 6 - 9: Number of tourists, including pilgrims

Row number in figure 6-9	Distance travelled
3	Less than 100 km
4	100 km - 500 km
5	500 km - 1000 km
6	More than 1000km

Table 6 - 10: Distances travelled by tourists

Although the answers given have to been seen as rough estimates, they show a very good correlation with the number of tourists in total. This means that the higher the amount of tourists is, the higher is the percentage of people travelling larger distances. The results are summarized in figure 6-9.

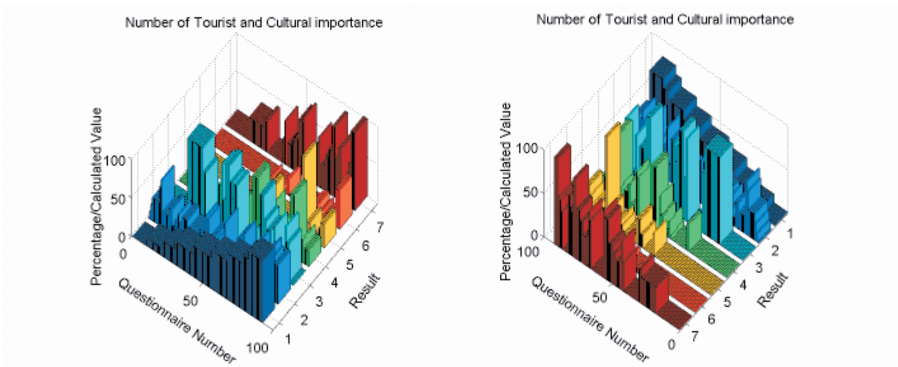


Figure 6 - 9: Tourists and cultural importance

For a better understanding the same plot is shown from different viewpoints. The left picture shows the front view. On the contrary, the right picture is the view from the rear. Seven rows are plotted, which explain the results for each of the 92 questionnaires. The questionnaires are sorted according to the responses of table 6-9 in ascending order, which are plotted in row number one.

The second row plots the subjective reputation of the church as analysed in table 6-8. Rows three to six visualize the percentage of tourists travelling what distance, where the number of rows is given in table 6-10. Finally, the seventh row shows the results of a rough calculation reflecting the distance travelled and the number of visitors with formula 6-1.

$$row\,7 = \frac{100}{\max_{row\,7}}(\%_{row\,3} \cdot 1 + \%_{row\,4} \cdot 2 + \%_{row\,5} \cdot 3 + \%_{row\,6} \cdot 4) \cdot row\,1 \tag{6.1}$$

The percentages of each row are multiplied with a simple importance factor and added. Then they are multiplied by the total number of tourists and normalized to a maximum of 100. In figure 6-9, it can be seen that the subjective assessment of the reputation does not correspond at all to the number of tourists and even less to the cultural importance, if the travel distance is regarded by formula 6.1. For the assessment of the cultural importance it can be concluded that the travel cost method – even if reduced to formula 6.1 – provides the best results.

To assess the human risks, the maximum number of persons being in the church at the same time was evaluated in a next step. For the calculations in this chapter, it is assumed that this number is mainly caused by single religious or cultural events and not by the amount of tourists in the church. This study also included information about how often this number is reached. The results of the two questions are presented in the following table.

Maximum number of persons	**Percentage**		**How often is the number reached**	**Percentage**
Less than 30	6.6		Once	8.7
30-50	0.0		Twice	4.3
50-70	2.2		Three times	4.3
70-150	9.8		Four times	9.8
150-300	33.7		Five times	10.9
300-500	23.9		5-8	14.1
500-1000	18.5		9-12	5.4
More than 1000	5.4		More than 12	42.4

Table 6 - 11: Maximum number of persons in the church

Next to the maximum number of persons in the church, it is important to know the mean number to predict the mean risk and its possible dispersion. Concerning the mean number of persons in a church, the following statements may be derived from the responses.

- Around 6.6% say that the mean number is below 20
- 1 answer (1.1%) was that the mean number is between 20 and 50
- 11.96% estimate the mean to be between 50 and 75
- A number between 75-100 is the average for 15.2%
- 14.1% state the mean between 100 and 150
- 16.3% answer with 150-250
- Between 250 and 400 participants are the average for 21.7%
- Finally, 13.0% have more than 400 visitors as a mean

Finally, the last point having an important influence on the human risk is the number of religious events held in the church. This might differ significantly for weekends and weekdays, as the answer to question 9 in the field of human risk revealed. This information is plotted in figure 6-10. These data, as all others in this study, were evaluated only for catholic churches, but they should be transferable to other churches as well as to mosque and synagogues.

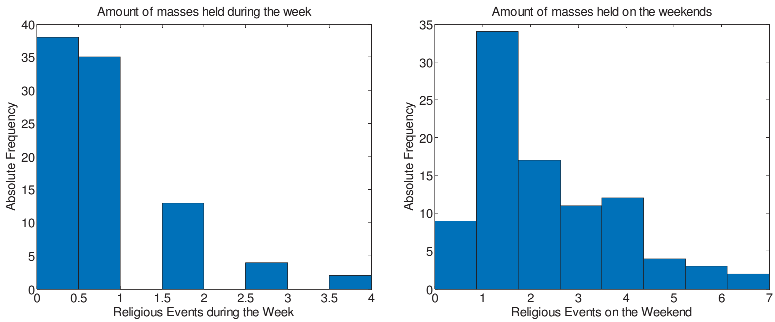


Figure 6 - 10: Frequency of the amount of religious events held within the week (per day) and on the weekend (total number)

The collected data provide sufficient background to compare the human risk of churches. If the probability distributions shall be analysed more precisely, the seasonal deviations of tourists have to be included in the study. Both [BENICULTURALI 2006] and [DESTATIS 2005] provide helpful information. [BENICULTURALI 2006] gives information about the number of visitors to museums, monuments and cultural circuits distributed over the year. All resemble each other and are plotted in figure 6-11. Having collected all this information, it is possible to derive artificial density distributions of the number of persons in the church. Therefore, a short program was written, which is also attached in Appendix I. The following input is necessary or possible:

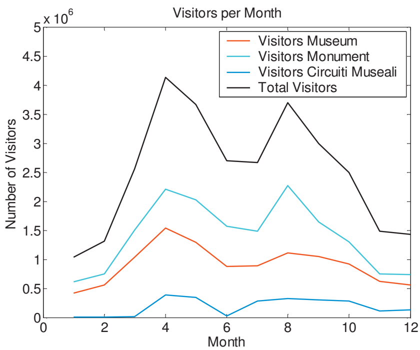


Figure 6 - 11: Distribution of visitors throughout the year

- The total number of inhabitants, assuming that they spent the night in the church or directly adjacent to the church
- The number of workers, assuming that they spent 10 hours a day working on or in the structure
- The mean number of visitors throughout the year and their standard deviation, which, if not known, can be estimated as 0.1 times the mean
- The mean number of persons visiting religious events on weekends and their standard deviation, which can be estimated as 0.2 times the mean
- The mean number of persons visiting religious events on weekdays and their standard deviation, assuming – if not known – that it is equivalent to 0.2 times the mean
- The number of religious events held on weekdays and on Sundays
- The maximum number of people in the church and the information how often it is reached, assuming a standard deviation of 0.1 times the mean if not known better

After the matrix is filled, each coefficient is multiplied with a random number based on a standard normal distribution with the given coefficient of variation. Seasonal distribution is adjusted according to monthly factors derived from figure 6-11. A sample input for a church including visitors and several religious events is given in table 6-12. The resulting output is shown in figure 6-12 and 6-13.

Input	Value
Number of inhabitant	10
Number of workers	15
Mean of visitors	100
Standard deviation visitors	30
Mean weekend	160
Standard deviation weekend	40
Mean week	90
Standard deviation week	20

Input	Value
Number of events / week	5
Number of events / weekend	2
Maximum number of persons	200
Standard deviation of maximum	30
Number maximum is reached	6

Table 6 - 12: Sample input for assessing the human risks

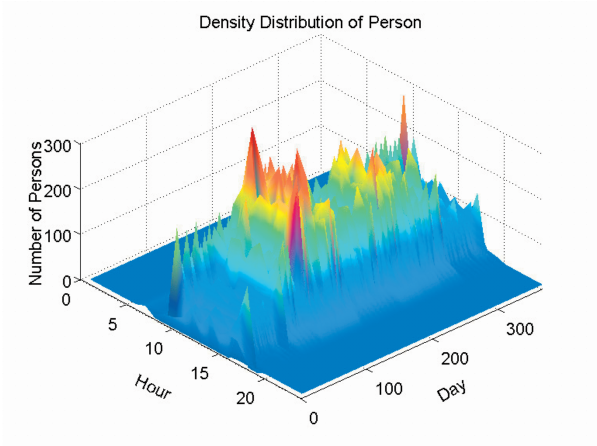


Figure 6 - 12: Yearly density distribution for a church according to input from table 6-12

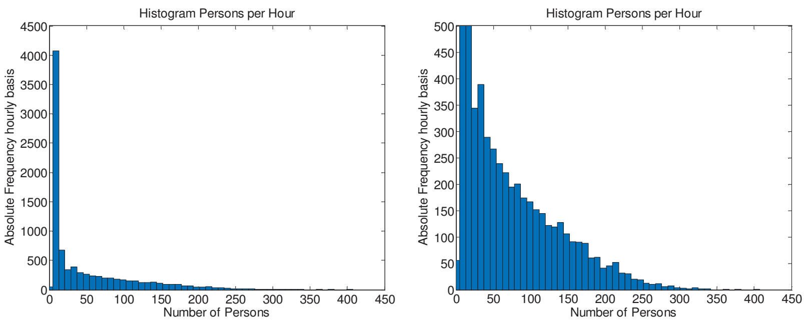


Figure 6 - 13: Results from figure 6-12 expressed as a histogram

Left handed in figure 6-13, the absolute occurrence of persons per hour is shown. Naturally, during large periods of the day only few people are inside the building. This is true also for normal office buildings, which are not occupied during the night or residential buildings, which are empty during large parts of the day. The right plot of figure 6-13 gives the same results, but for better visibility the y-axis is limited to 500. The plot shows a steady decrease in numbers. However, this cannot be concluded for all cases. Depending on the relation between visitors and participants in religious events, there may be small peaks for higher numbers. Thus, a common probability density function cannot be assigned. The usefulness of this program will be explained in chapters 6.7.2, 6.7.4 and 7.

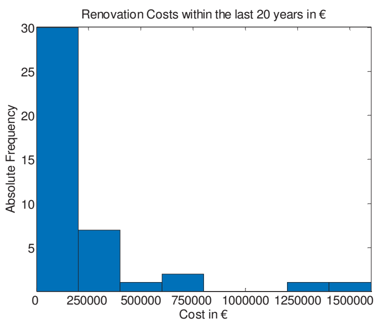
To round up the data on human risks and to include the social importance of the church, the last question dealt with the size of the religious community in square kilometres. The response is listed in table 6-13. Originally, the results were supposed to serve as additional information about the social risk, but the area could not be connected to size or percentage of visitors, thus it is not used further.

What area is covered by the church	Percentage
Less than 1 km²	28.2
1-3 km²	25.0
3-5 km²	23.91
5-10 km²	16.31
10-15 km²	3.3
15-20 km²	3.3
20-50 km²	0
More than 50 km²	0

Table 6 - 13: Area of religious communities in the questionnaire

6.5.2.4 Financial and economic situation

While the response and the evaluation of the human risk can certainly be regarded as reliable and very informative, the financial and economic results are less useful. Originally, the idea was to collect data on the number of employees, costs, insurances and incomes to create a full overview of the economical situation of churches. The first test has shown that most people were unwilling to offer information or were not fully sure about how to answer the financial questions. As a consequence, only the yearly costs spent for renovation and maintenance and large costs within the last 20 years were evaluated to relate them to the costs of life quality indices. Results illustrated in figure 6-14 show the large scatter on the one hand and sometimes extremely high costs on the other, which would probably justify higher expenses for strengthening the structure for high risk. Nevertheless, the information could neither be verified by other sources, nor is it assumed to be reliable, since in some cases the responses were added by a question mark. This indicates that respondents were not always sure about the correct answer. Since very satisfactory results could already be obtained on the other topics, there was no need to include these unstable results in the analysis. Financial losses are thus not regarded further in this work, although this would be desirable, because – if available at a central location – the risk assessment would benefit.



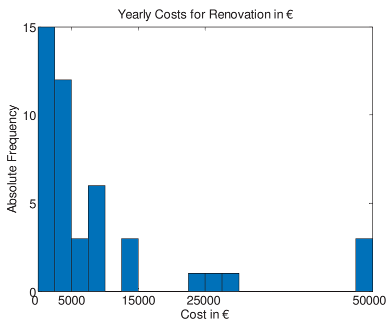


Figure 6 - 14: Renovation costs

6.5.2.5 Structural situation and previous damages

Relevant data was gathered to see whether the general structural condition is good or bad and to evaluate how often churches suffered damage in an earthquake event. The results were quite interesting in their dimensions. In Italy 33 out of 47 churches have suffered damage due to extreme events, which is equivalent to 70% of the structures. 26 (55%) were damaged by earthquakes, three (6%) by flooding and four (8%) by extreme winds. In Germany, 12 out of 45 churches (27 %) suffered damage from settlements, which were the major cause for damages. It is likely that Italian churches suffered also from settlements, but the damages were superposed by other extreme events. It is also remarkable that 10 German structures, which equals 22%, were damaged by earthquakes. In one case damages occurred, when a lightning struck the tower of a church.

6.5.2.6 Geometrical layout

From the geometrical data it can be concluded that in Italy most common churches exhibit a flat roof or a barrel vault in the main nave. Often, i.e. in 19 out of 47 answers, they consist of only one nave, which is accompanied by a single dome in the crossing. In Germany the typical church is characterized by three longitudinal naves, each consisting of mostly 5 repeated structural elements and one abside. The response to the geometrical question is listed below.

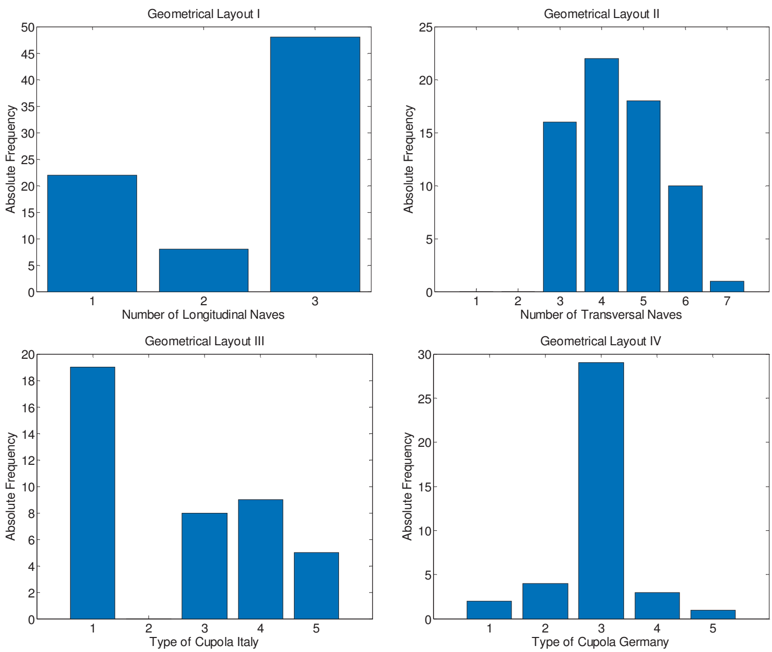


Figure 6 - 15: Responses to geometrical questions

The following table comments once more the type of cupola by relating the numbers of figure 6-15 to the type of vault.

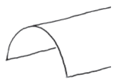
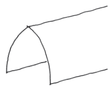
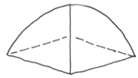
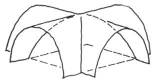
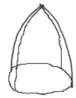
Vault No. 1	Vault No. 2	Vault No. 3	Vault No. 4	Vault No.5
				

Table 6 - 14: Different types of vaults

6.5.2.7 Miscellaneous

Nearly all participants desired to be informed about the results of the thesis. No criticism was stated. Instead, the blank space at the end was in some cases used to offer more information about the church, which can be regarded as a sign for the interest of the participants.

6.5.3 Discussion of the results

The questionnaire was in most parts a full success. Out of the six goals which should have been accomplished, only the estimates of reconstruction costs could not be provided. Especially the evaluation of human risks can now be carried out very efficiently, by assessing the probability density of persons inside the church with the appended program and comparing it to other churches or structures. Also, the collected data are representative enough to serve as a background for the assessment and comparison of social, historical and cultural losses. General characteristics and the susceptibility to damages due to the surroundings have also been highlighted, offering interesting information about the types of hazards imposed on structures with a long design service life.

The next step will now be the comparison of the results with the purpose to grade them into risk classes. This will be explained in the following for human and CSH risks. Due to the lack of reliability of the financial data, these risks are not included in this study. One reason for this is that reconstruction and renovation costs may vary largely depending on the region, the type of structure and the type of damage. Another reason can be seen in the statement that the economic risk should include possible losses due to a decreasing number of tourists visiting the place and the stores and offices located nearby. The results of the survey do not permit to draw any conclusions on this matter. For all theses reasons, economic risks are excluded from this study and remain a task of research.

6.6 Creation of risk classes

6.6.1 Human risk

Human risks may be evaluated in a two step procedure. At first, typical constellations of visitors and number of participants in religious events have to be derived. With these data, which were already presented in chapter 6.5.2.3, density distributions as plotted in figure 6-12 may be derived.

When this is done, it can be easily seen that large differences exist between small churches serving only a small community and large monuments with large numbers of tourists, which are also used for the celebration of religious events. Several subgroups could be created between these two extremes. The density distributions, as shown in figure 6-12, are dominated by the number of religious events and persons joining them as well as by the number of tourists visiting the church. Both are slightly correlated, which signifies that the higher the number of tourists is, the more persons are usually also attending religious services. As a first step, churches with a very low potential of loss of human life may be evaluated. This is true for very low numbers of persons and tourists. The same task can be performed for the maximum risk taking the maximum values of visitors and participants into account. Those cases are taken as exceptional. The majority of all churches should fall somewhere in between.

To derive risk classes, as they are shown in tables 6-15 to 6-18 and table 6-20, typical values of the response analysis must be used. The presented risk classes are based on figures 6-5, 6-7, 6-17 and 6-18. It is assumed that – apart from the outer classes 1 and 5 – all categories have the same probability, i.e. approximately 30%, and that the mean value of all churches falls into category 3. This is an useful assumption and proved reliable, although other approaches could also be discussed. Tables 6-15 and 6-16, which are based on the calculation of 100 artificial density distributions of persons, present four different possibilities to derive a human risk class. For applications, the final risk class should always be the maximum one.

Risk Class	Mean number of mass participants	Yearly number of tourists
1	Less than 30	Less than 100
2	30-100	100-500
3	100-180	500-5000
4	180-500	5000-50000
5	More than 500	More than 50.000

Table 6 - 15: Proposal for simple determination of risk classes

Risk Class	Mean number of persons per hour	Standard deviation hourly number of persons
1	0-3	~ 5
2	3-10	~ 12
3	10-35	~ 30
4	35-100	~ 60
5	Above 100	~ 120

Table 6 - 16: Risk classes fitted according to table 6-15

6.6.2 *Historical values*

When assessing the historical value of the church, the construction year of the church is the only parameter regarded. Cultural goods, artefacts, wall paintings or the social importance are not at all considered in this context. Certainly, to regard merely the construction year is an approximation. The surroundings, the completeness of the building, changes to structure and material should be considered as well, to name only a few parameters which are linked to the historical value. It is thus recommended never to use historical values alone, but only within the triptych of CSH values. The same holds for social and historical values. As will be explained in chapter 6.6.5, personal preference factors will be introduced, which allow the slight modification and amplification of the factors, if subjective assessment requires to value for example the outstanding original completeness of an architectural ensemble. In this way, the construction year may be used as the simplest measure for the assessment of the historical risk.

The mean construction year of all churches considered in Germany is 1795, as shown in figure 6-7. For those built before 1945 the mean is given as 1690. The latter value is taken as more representative, since it considers a larger number of samples in comparison to those of the questionnaire and does not include recently built churches. It is important to determine a year before which the structures are assumed to be classified as “historical”. This date has a large impact on the mean value of the construction year, which in turn has significant influence on the outcome of the overall historical risk as will be seen in the following. The mean age gives good reference to the implementation of risk classes. The following proposal is based on risk classes, which were derived according to the procedure described in chapter 6.6.1 for all churches considered in Germany and in the questionnaire, assuming that all churches constructed before 1950 are categorized as historical.

Risk Class	**Construction Year**
1	1900-1950
2	1800-1900
3	1600-1800
4	1000-1600
5	Before 1000

Table 6 - 17: Proposal for historical risk classes

The historical importance might also be expressed in terms of a formula. This parameter, in the following called *Historical Importance Factor* or *HIF,* is set to one for the mean value of all churches considered. Assuming an exponential distribution for the age distribution as seen in figures 6-7 and 6-8 the HIF may be expressed by the following formula.

$$HIF = \frac{\mu \cdot e}{(A_h - \mu)} \cdot (A_h - x) \cdot \frac{1}{\mu} e^{-\frac{x}{\mu}} \qquad (6.2)$$

Where HIF is the Historical Importance Factor, μ the mean age of all churches considered, A_h the year at which the church is assumed to be historical, x the age of the considered church. The first ratio is a factor determining that at the mean age the HIF is one, the second factor considers the age. The purpose of the third is to assign a true value of the HIF based on the assumption of an exponential distribution. Simplified, the formula may be rewritten as:

$$HIF = \frac{(A_h - x)}{(A_h - \mu)} \cdot e^{(1-\frac{x}{\mu})} \tag{6.3}$$

Assuming that a church is historical, if it was built before 1945 and a mean construction year of all considered churches of 1690, the HIF for two different churches build in 1880 and 1200 is determined as follows.

$$HIF = \frac{(1945 - 1880)}{(1945 - 1690)} \cdot e^{(1-\frac{1880}{1690})} = 0.23 \tag{6.4}$$

$$HIF = \frac{(1945 - 1200)}{(1945 - 1690)} \cdot e^{(1-\frac{1200}{1690})} = 3.90 \tag{6.5}$$

The value of the HIF is presented in figure 6-16. Its maximum value depends largely on two values. At first the mean construction year and secondly the age of the oldest church considered in the sample study. The drawback of this approach can be seen in very high HIF values for older structures, if the mean construction year is above 1700. Those values could only be justified if the distribution of the construction would truly be exponential, which is not the case. The problem will be addressed once more in chapter 6.6.5 including the presentation of a general solution.

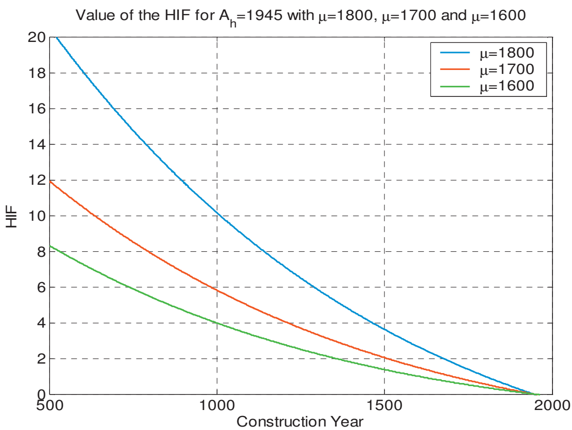


Figure 6 - 16: Value of the HIF for different mean construction years

6.6.3 *Social values*

The social values describe the importance for making social contacts, meeting and the time purposely spent with other persons. This naturally includes the religious events. Therefore, social values may be measured by the time spent for social events happening in or around the church of which the celebration of religious ceremonies itself is certainly the major contribution. The data considered in the questionnaire included only masses. Other events, such as weddings, baptisms, birthdays, prayers, festivals, etc. are not included. They are normally depending on the size of the religious community belonging to the church.

To evaluate the social value, it is thus proposed to include both values, the mean number of persons participating in the masses as a measure for the active community and the size of the religious community to take into account passive effects, such as weddings or others named before. Additionally, the importance of the church for the community is included by capturing the percentage of persons visiting normal religious events to reflect the degree of religious activity. The two latter factors are multiplied by the mean number of visitors of all churches (NP_{mean}) and divided by the mean of all communities ($S_{c,mean}$ and P_{mean}).

$$SIF = \left(NP + \frac{S_c}{S_{c,mean}} \cdot NP_{mean} + \frac{P}{P_{mean}} \cdot NP_{mean} \right) \cdot \frac{1}{SIF_{raw}} \tag{6.6}$$

Where:

SIF	=	*Social Importance Factor*
NP	=	Mean number of participants of religious events in the considered structure
NP_{mean}	=	Mean number of participants of all comparable structures
S_c	=	Size of the religious communities in units of persons
$S_{c,mean}$	=	Mean size of the religious communities of all considered churches
P	=	Percentage of persons taking part in the religious event
P_{mean}	=	Mean percentage of all considered churches

Within the second and third factor, NP_{mean} was used instead of NP, because NP favours large churches with a large number of visitors, albeit they might have a low percentage of persons really visiting the masses. NP_{mean} enables a more comparative approach. If it was not included, churches with a large community would dominate the result. SIF_{raw} is the mean value of the SIF for all considered churches, calculated as follows.

$$SIF_{raw} = \frac{1}{n} \sum_{i=1}^{n} NP_i + \frac{S_{c\,i}}{S_{c,mean}} \cdot NP_{mean} + \frac{P_i}{P_{mean}} \cdot NP_{mean} \tag{6.7}$$

Again, the mean value of all results is set to one. Giving the results from the diocese of Aachen, the Social Importance Factor for all churches is calculated and the results shown in the next figure are obtained. The value for SIF_{raw} in the diocese of Aachen was calculated to be 673.

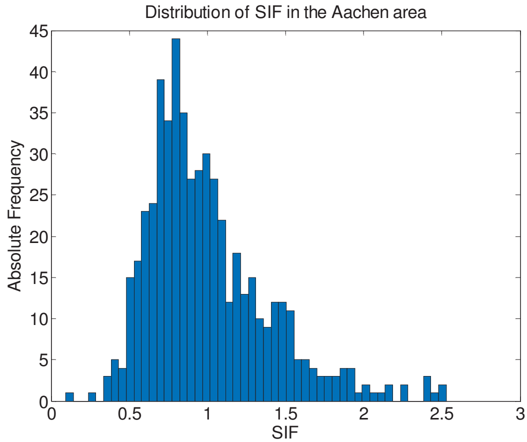


Figure 6 - 17: Social Importance Factor for all churches in the diocese of Aachen, Germany

With respect to the diagram above, the following risk classes for the cultural importance may be derived. Again, the first and the last class intend to cover extreme values, whereas the three classes in the middle cover areas with equivalent probability.

Risk Class	SIF Factor
1	Below 0.5
2	0.5-0.8
3	0.8-1.2
4	1.2-2.0
5	Above 2.0

Table 6 - 18: Proposal for social risk classes

6.6.4 Cultural values

With the HIF and the CIF two factors were introduced, which allow an easy comparison of different churches. Now, the same has to be done for the evaluation of cultural goods. As explained before, it was not possible to conclude factors based on the number and description of cultural goods in the church. Neither were the subjective descriptions of the churches useful. What is possible instead is to rely on approaches used in cultural management and heritage economics. Data of tourists were sufficient to use them as a background for grading the cultural values. Originally, the use of the travel cost approach would require the evaluation of the formula:

$$CV = \sum_{i=1}^{n} T_i \cdot h_i \qquad (6.8)$$

Where CV is the cultural value based on the amount n of tourists T, travelling h hours to see the building. The travel time is especially important, because, if the church is not located within a larger city but a smaller village, its cultural importance will be evaluated higher. In application, the CV is often hard to determine, since reliable information about real travel time is based on estimates. Thus, it is proposed to group the tourists according to rough distances they have travelled. To do so, the information of the questionnaire is used once again. With the classification of table 6-19 it is possible to calculate the cultural importance with formula 6.9. This formula is supplementary for the full travel cost approach, which could be used instead, if sufficient data is available.

Distance	Group
Less than 100 km	1
100-500	2
500-1000	3
More than 1000 km	4

Table 6 - 19: Groups used in formula 6.9

$$CIF = 0.3 + \frac{NT}{CIF_{raw,mean}} \cdot \sum_{i=1}^{4} P_i \cdot i \qquad (6.9)$$

With *CIF* being the *Cultural Importance Factor*, NT the number of tourists and pilgrims, which does not include persons belonging to the religious community, P_i the percentage of tourists belonging to group i as shown in table 6-19. The $CIF_{raw,mean}$ is introduced because otherwise a high number of tourists would result in extremely high CIF values. It is the mean value of all considered churches, at best on a national level and calculated with formula 6.10.

$$CIF_{raw,mean} = \frac{1}{n}\sum_{i=1}^{n} \left(NT \cdot \sum_{i=1}^{4} P_i \cdot i \right) \qquad (6.10)$$

The $CIF_{raw,mean}$ of this study was determined to be 1 330 000. This value may be used if no other information is available. The CIF is solely based on the number of tourists. Still, most persons will agree that although no tourists will visit the church, the church should still have a cultural value. This is one reason why the minimum CIF value was set to 0.3 in formula 6.9. Otherwise the CIF only ranks very high numbers of tourists. The distribution of the CIF for the diocese Aachen in Germany is presented in the next figure. According to the approach described before, the cultural risk classes listed in table 6-20 may be derived from the distribution plotted in figure 6-18.

Risk Class	CIF Factor
1	0.3-0.35
2	0.35-0.5
3	0.5-1.0
4	1.0-5.0
5	Above 5.0

Table 6 - 20: Proposal for cultural risk classes

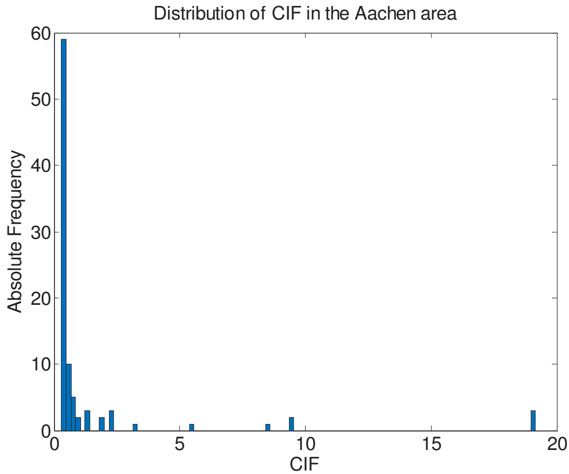


Figure 6 - 18: Distribution of the CIF in Aachen, Germany

6.6.5 Proposal for linking the CSH values

Cultural, social and historical values are all intangible values. For reasons of simplicity and better application they should be summarized into a single measure. The resulting number would also enhance the overall risk assessment, since very high or low results achieved in any single value of the CSH are smoothened as the three different results are added. To do so, all values should fall within a comparable range. At this stage, this cannot be guaranteed although the range of the three variables is within the same order of magnitude. It is thus proposed to adjust the values in such a way that they fall in the range between 0 and 10. This number is arbitrary chosen, but with respect to the values defined for the CIF, SIF and HIF, it is appropriate. Again, they should be adjusted, so that the mean value of each factor is equal to one. By these means, a better differentiation may be provided for high risk structures, which are of greater interest.

The connection of the three values may flatten some of the drawbacks of the single factors explained in the previous chapters. With respect to the HIF, it was for example stated that it governed by the mean construction year of all churches and by the oldest church in the sample. This may result in exaggerated values of the HIF for very old churches, since an exponential distribution was assumed, which is not justified for construction years before 1000. Reflecting this matter and including the result that the mean construction year is close to 1700 and the term historical refers to churches built before 1945, the proposal of figure 6-19 for a general HIF depending on the construction year is offered.

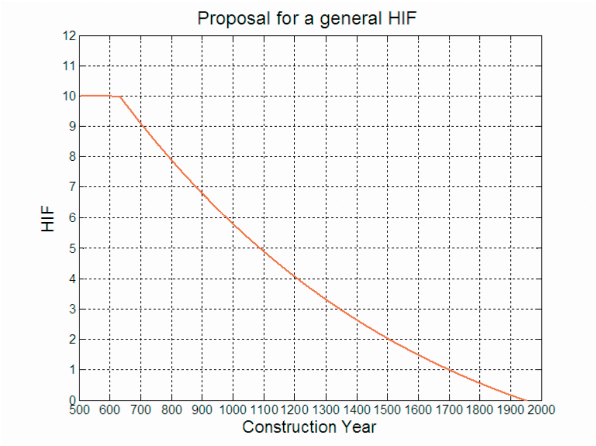


Figure 6 - 19: Proposal for a general HIF

The mean value of 1700 corresponds to a HIF of one. The graph is similar to the line for a mean of 1700 shown in figure 6-16. The only difference is that it is flat for very old structures, assuming, that a HIF of 10 is never exceeded, which corresponds better to the results of the questionnaire plotted in figures 6-7 and 6-8.

For the HIF, values above the proposed range from zero to ten were calculated. SIF values range between zero and 2.5. Even for churches with a large and active religious community, values of 2.5 have not been exceeded. To expand the maximum to a value of ten, the calculated SIF must be multiplied by a conversion factor. Still, the mean value of all churches has to remain close to one. Therefore, if high values are multiplied, lower ones must be decreased, so that the mean remains one.

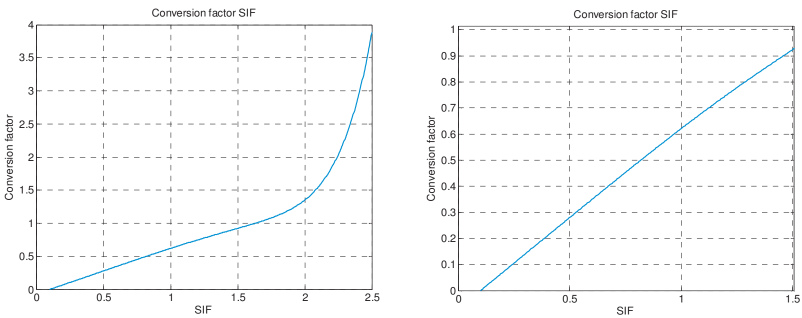


Figure 6 - 20: Proposed conversion factor to transfer SIF into the range from 0 to 10

This can be done by applying the conversion factor presented in the previous figure. If the SIF from figure 6-17 is multiplied by the conversion factor plotted in figure 6-20, the following distribution for $SIF_{refined}$ is obtained.

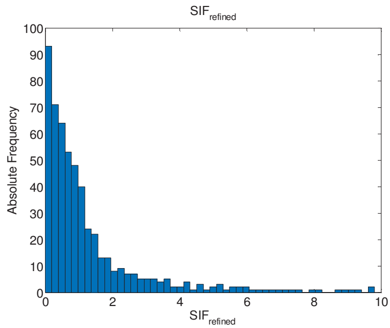


Figure 6 - 21: Distribution of SIF$_{refined}$

The conversion for the CIF is performed in the same manner. The results are given in figure 6-22. The author is aware that the conversion factors are crude approximations and certainly, applications have to prove whether the proposal may be accepted or not. Nevertheless, all factors are treated as equal by this method and a comparison of risks is more precise than it could be done before.

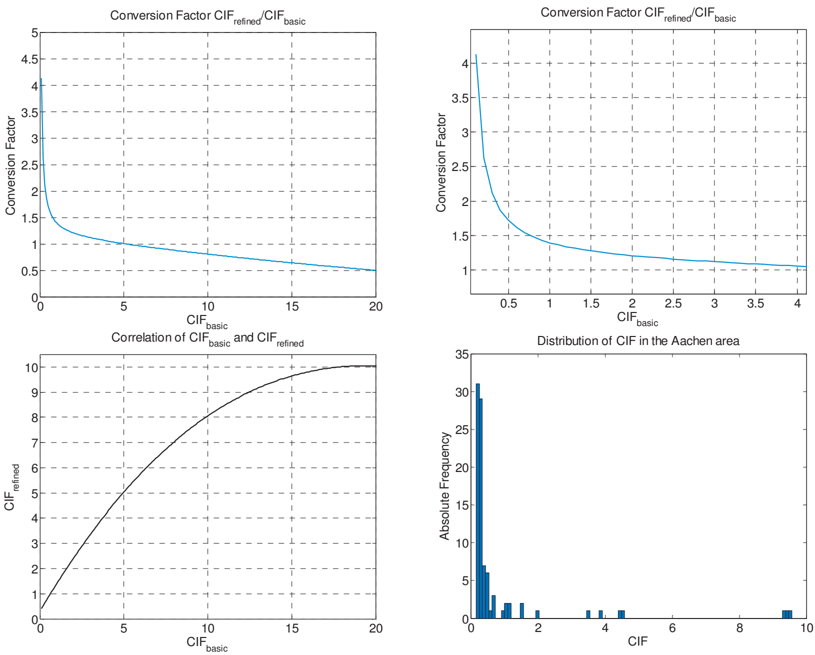


Figure 6 - 22: CIF conversion factor - upper line, relation CIF and CIF revised - lower line, left side, CIF distribution in the diocese Aachen - lower line, right side

After this is done, all three importance factors lie in the range between zero and ten, all have a mean value of one and all exhibit an exponential decline. The total value of the CSH might now be determined as follows:

$$IF_{CSH} = \alpha_{P,CIF} \cdot CIF_{refined} + \alpha_{P,SIF} \cdot SIF_{refined} + \alpha_{P,HIF} \cdot HIF_{refined} \tag{6.11}$$

Where IF_{CSH} is the importance factor of the total CSH values and $\alpha_{P,i}$ are the preference factors for the subjective modulation of the data. The preference factors should not be below 0.5 and should not exceed the value of 2. They should only be applied with great caution and preferably only one value should be increased. Also, the value of each factor cannot exceed the maximum value of ten. Consequently, the maximum IF_{CSH} value of 30 – or 40, if one preference factor is applied – cannot be exceeded. Since the mean for each single value equals one, the mean for IF_{CSH} is three. Typical IF_{CSH} values should therefore fall into the range between zero and five. The preference factors are for example applicable, if one church has a very unique construction style which – for some reason – is unique among all churches considered in this study. In that case $\alpha_{P,HIF}$ might exceed the value of one.

6.6.6 Critical review and summary

Based on a data comparison, the human risk of churches could be evaluated objectively and five risk classes were derived as a summary of the main characteristics. Describing the intangible CSH losses is a task not being solved easily. The data gathered were used to construct three independent descriptors of the intangible risks. Furthermore, the absolute occurrences of the importance factors for a sample study of 537 churches were calculated and risk classes were derived. In the next step, the three importance factors were modified in such a way that all had similar characteristics concerning the range of values, mean and type of distribution. In this way, it is possible to combine all values, by adjusting them slightly with subjective preference factors.

Now, although a preference factor is introduced, strict regulations are given for its application. In this way, slight adjustments can be made, if a really extraordinary situation occurs. Otherwise, the importance factor IF_{CSH} provides a good description of the intangible risks. Since the results of a complete diocese were used, results are transferable and representative for other cases, although on a national scale a more detailed comparison should be performed.

When using the importance factors, it must be realized that they do not intend to assign a single absolute value to a church. Instead, they should be used as a parameter, which supplies a specific characteristic to compare similar buildings of the same region. Its purpose is to create risk classes, derive target reliabilities and to determine highly exposed structures.

6.7 Determination of the overall loss potential

6.7.1 Transferring the values into years of human life lost

In chapter 2 the most common risk measures were explained. It was easy to see that all of them reflect either economical values or the loss of human life. The loss of human life itself may be described by different measures from simple Mortality Rates over Fatal Accident Rates to

Years of Life Lost or Lost Life Expectancy. Terms used for the evaluation of optimal safety investments were introduced, such as NCAF, the Net Costs of Averting a Fatality and SVSL, the Societal Value of a Statistical Life. As the two most powerful tools to describe the relation between acceptable risks, the P-D diagrams and Fatal Accident rates were identified. All measures have in common that they include the loss of human lifetime or the time spend in performing some activity, be it working, relaxing, living, or something entirely different. CSH values are not included in any of these descriptors.

The data assembled in this study can now be used to fill this gap. Certainly, it cannot be the task of one individual to provide the final answer to this matter, which deserves detailed discussion within a large community. Still, a tentative proposal may be given, which transfers the CSH-values into loss of human lifetime. Since it is difficult to define an absolute parameter for the CSH-values, upper and lower bounds will be used. The real value would lie somewhere in between. The lower boundary is derived easily, if the times being spent in the church listed for example in figure 6-13 are summarized over the year. For the plot shown in figure 6-13, a total of 424860 hours would be obtained. This corresponds to 48.5 years. If the church is closed as a result of an earthquake for one year, the minimal loss of social and historical values would be equivalent to 48.5 Years of Life Lost.

Years of Life Lost may be expressed as human lifes lost by dividing the total amount of Years of Life Lost by the difference between mean life expectancy and mean population age. Table 6-21 includes some information about the mean life expectancies in several countries according to [HDI 2001]. The mean population age is usually provided by the national statistical offices. In Germany the mean population age is 43 years [DESTATIS 2005]. If the church collapses, it is proposed to multiply this value with the design service life of the structure, which is given with 100 years for monuments in [EC 1]. Since it is not the remaining design working life, which is addressed, and predictions into the far future are seldom reliable, this work uses the lower value of 50 years. Now, if the church collapses completely, the loss of social and historical values would correspond to the following loss of human life:

$$\frac{50 \quad (\text{years closed}) \cdot 48.5 \quad (\text{human lifes lost/year})}{77.6 \quad (\text{mean life expectancy}) - 43 \quad (\text{mean population age})} = 70 \text{human lifes} \tag{6.12}$$

Human Development Index Rank	Country	Mean Life Expectancy
1	Norway	78.4
6	United States	76.8
17	Germany	77.6
20	Italy	78.4
58	Romania	69.8
83	Turkmenistan	65.9
120	Lesotho	47.9
162	Sierra Leone	38.3

Table 6 - 21: Mean life expectancy [HDI 2001]

To calculate the upper bound, other parameters must be included. Therefore, the travelling time to the church will be included by a lump sum of 25% of the total time spent in the building. A value of 25% was chosen, because longer tourist travels include several visits to other places and the time spent walking or driving to a regular religious event does usually not exceed this value. Additionally, the loss of historical value will be integrated. For damage category 2, the years of human life lost due to the loss of historical value is assumed to be 1% of the building age, damage category 3 will be accounted for with 2% loss of the building age. These factors intend to grasp approximately the damage to the building. If a damage category 4 or 5 occurs, parts of the building have already collapsed. The loss of historical value is in these cases measured by the percentage of collapse of the complete structure, multiplied with the age of the building. Overall, these are plausible approaches, which could be specified further. A church build in the year 1246, which collapsed during an earthquake in 2006, would consequently result in 760 YLL. For any structural parts that were added afterwards, the reference value will still be the original date of construction, if the structural integrity has not fully changed. The contribution of the historical values is less dominant in the assessment of churches receiving high numbers of visitors, whereas for low number of tourists its importance increases. With this done, it can be easily seen that while the approaches described in chapter 6.6 may be used for the grading of churches into risk classes, it is now possible to compare different kinds of risks, such as sport or health risks to earthquake risks of historical structures. The following chapter will illustrate the previous descriptions by an example.

6.7.2 Implementation into P-D diagrams

With the explanation given so far, it is possible to include these data into P-D diagrams. This is best illustrated with an example. Considered will be a church with the following assumptions.

- Damage grade 1 takes place with a probability of 10^{-1}
- Damage grade 2 takes place with a probability of $5 \cdot 10^{-2}$
- Damage grade 3 takes place with a probability of $5 \cdot 10^{-3}$
- Damage grade 4 takes place with a probability of 10^{-4}
- Damage grade 5 takes place with a probability of 10^{-5}
- Damage grade 4 is subdivided into the following probabilities
    - 10% of the vaults collapse with a probability of 10^{-4}
    - 25% of the structures collapses with a probability of $5 \cdot 10^{-5}$
    - 50% of the structure collapses with a probability of $2 \cdot 10^{-5}$
- First finial collapses with probability of $5 \cdot 10^{-3}$
- The distribution of persons in the follows figures 6-12 and 6-13
- The summation of the time persons spent in the church is equal to 48.5 years
- The spatial distribution of persons in the church is even, meaning, if 10% of the structures collapses, 10% of the persons inside are affected
- The church was build in 1520 and is assessed in 2006
- Economic contributions are not evaluated
- Mean age in Germany is 43 years
- 1 human life lost corresponds to 34.6 Years of Life Lost
- Mean number of persons in the church is 48.5 with a probability of exceedance of 0.31

The histogram of the distribution of persons presented in figure 6-13 might also be expressed in CDF or probability of exceedance as shown in the following figure.

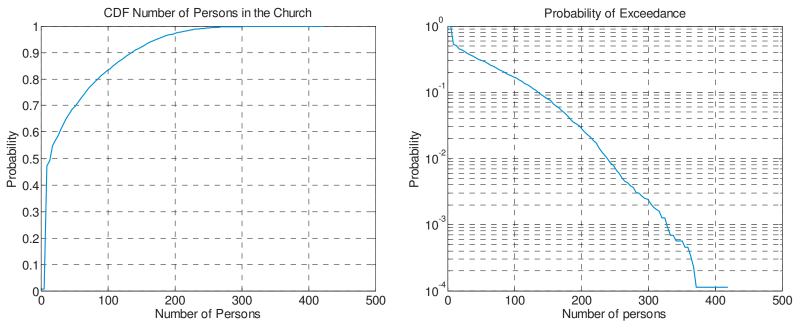


Figure 6 - 23: CDF and probability of exceedance of the density distribution presented in figure 6-13

The resulting Probability-Damage diagram, which is shown in figure 6-24, is obtained by the calculations given in table 6-22.

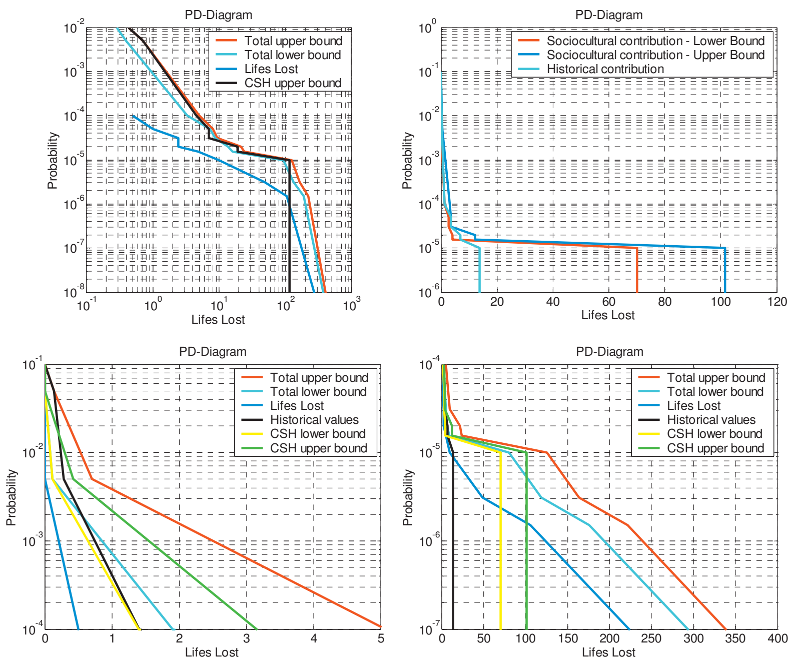


Figure 6 - 24: Resulting P-D diagram in various plots

Point	Damage Grade	Consequence	Description of Consequence	Calculation	YLL	Probability
1	1	No consequence	Not important for risk assessment	0	0	10^{-1}
2	2	Historical	1% loss - flat	(2006-1520)·0.01	4.8	$5 \cdot 10^{-2}$
3	3	Historical	2% loss - flat	(2006-1520) ·0.02	9.7	$5 \cdot 10^{-3}$
		Social/Cultural	Closed for repair for one month	48.5/12	4.0	$5 \cdot 10^{-3}$
4	***	Human	Partial collapse	1·34.6	34.6	10^{-6}
5	4	Historical	10% collapse vault	(2006-1520)·0.1	48.6	10^{-4}
		Social/Cultural*	Closed for 1 year	48.5·1	48.5	10^{-4}
		Social/Cultural**	Closed for 1 year	48.5·1.25	60.6	10^{-4}
		Human****	Fall of small plaster 5% of persons affected	48.5·0.05·34.6	84	$0.31 \cdot 10^{-4}$
6	4	Historical	25% collapse	(2006-1520)·0.25	121	$0.5 \cdot 10^{-4}$
		Social/Cultural	Closed for 2 years	48.5·2	97	$0.5 \cdot 10^{-4}$
		Social/Cultural	Closed for 2 years	48.5·2·1.25	121	$0.5 \cdot 10^{-4}$
		Human****	Fall of small plaster 10% of persons affected	48.5·0.1·34.6	167	$0.155 \cdot 10^{-4}$
7	4	Historical	50% collapse	(2006-1520)·0.5	242	$0.2 \cdot 10^{-4}$
		Social/Cultural	Closed for 3 years	48.5·3	145	$0.2 \cdot 10^{-4}$
		Social/Cultural	Closed for 3 years	48.5·3·1.25	181	$0.2 \cdot 10^{-4}$
		Human****	Fall of small plaster 20% of persons affected	48.5·0.2·34.6	335	$0.062 \cdot 10^{-4}$
8	5	Historical	100% collapse	(2006-1520)·1	480	10^{-5}
		Social/Cultural	Destroyed	48.5·50	2425	10^{-5}
		Social/Cultural	Destroyed	48.5·50·1.25	3031	10^{-5}
		Human****	Minimum number of persons in the church	10·34.6	346	10^{-5}
9	5	Human****	Mean number of persons in the church	48.5·34.6	1678	$0.31 \cdot 10^{-5}$
10	5	Human****	Mean plus 1 standard deviation	106·34.6	3667	$0.153 \cdot 10^{-5}$
11	5	Human****	Maximum number of persons in the church	423·34.6	14635	$0.1 \cdot 10^{-8}$

*For upper bound ** For lower bound ***Falling down finial kills one person with a probability of 10^{-6} ****Several points are usually obtained for the human risk, taking the product of probability of collapse and persons affected by the downfall of structural parts

Table 6 - 22: Calculation of the points of the P-D diagram

The diagrams are characteristic for most of the churches. Its main features are three different phases. At the beginning, for very low losses, the loss of CSH values is governing the total losses. This is described in figure 6-24 in the lower row. With the beginning of damage grade 4, the first loss of human life occurs and also the loss of CSH values increases. As a consequence the slope of the lines flattens. In this diagram damage grade 5 has a probability of 10^{-5}. At this stage the structure collapses completely. CSH losses cannot increase further, because the church is completely destroyed. In this last phase, the main contribution is given by the probability distribution of persons inside the church.

6.7.3 Implementation of the data into the LQI

Within this work, the description of the risk in terms of Probability-Damage diagrams is favoured. The Life Quality Index, which was already explained in chapter 2.3.7, is a tool more useful for the evaluation of strengthening methods and optimal investments into risk reduction measures. Thus, it is not extremely useful in comparing different risks. Still, some short comments are offered concerning possible applications of the collected data within the LQI.

If the mentioned data are to be included, two aspects would have to be regarded. At first, the increase in mean life expectancy should include the gain of life analysed for the CSH values. Additionally, w, the spare time could take into account either the full available time, or the time spent within the church solely. This is only a short comment, further research is necessary to connect the data to the LQI.

6.7.4 Comparison with other building types and other risks

As it can be seen in the P-D Diagram shown in figure 6-24, the total loss of life is nearly doubled, if the upper bound for CSH values is used. To analyse, how these data behave in comparison to other structures, two P-D diagrams for a sample residential building and a sample office building were also evaluated assuming the subsequent conditions.

	Residential Building	**Office Building**
Probability DG3	10^{-4}	10^{-4}
10% collapse DG 4	$5 \cdot 10^{-5}$	$5 \cdot 10^{-5}$
50% collapse DG 4	$5 \cdot 10^{-6}$	$5 \cdot 10^{-6}$
Probability DG 5	10^{-6}	10^{-6}
Density distribution of people	Cf. figure 6-25 left	Cf. figure 6-25 right
Mean number of Persons	2	50
Probability of mean number	0.5	0.45
Maximum number of persons	25	181
Probability of maximum number	10^{-4}	10^{-4}

Table 6 - 23: Assumptions for residential and office building

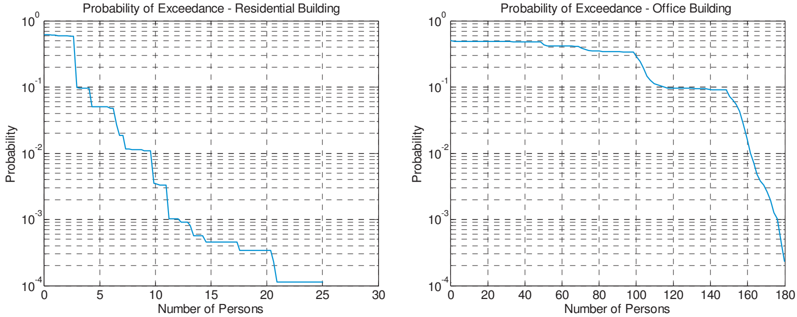


Figure 6 - 25: Density distribution of persons in a residential building (left) and an office building (right)

The different points of the P-D diagram may now be calculated according to table 6-22, without valuing cultural, social or historical values – which under given circumstances should also be regarded. The resulting differences between the church and the other buildings are described in figure 6-26.

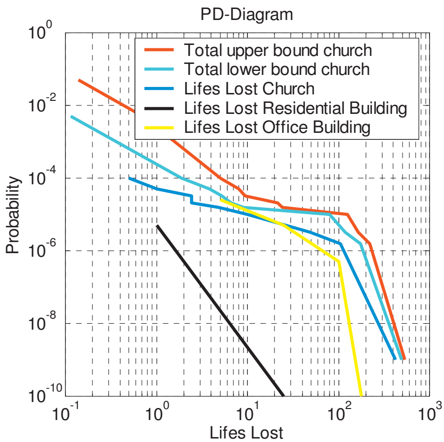


Figure 6 - 26: Comparison of residential and office buildings with churches

Although the results are only for one specific church, some typical characteristics of earthquakes risks of churches may be derived from the previous figure. At first, the results indicate that churches are more susceptible to hazards occurring at a lower frequency because of the contribution of CSH values. Secondly, for lower probabilities, i.e. below 10^{-5}, churches exhibit higher losses as a result of the high densities of persons in the church. Finally, parts of the lines of the office building and the church overlap, indicating a region of similar risk. The results might now be compared to diverse other kind of risks, e.g. those given in figure 6-27.

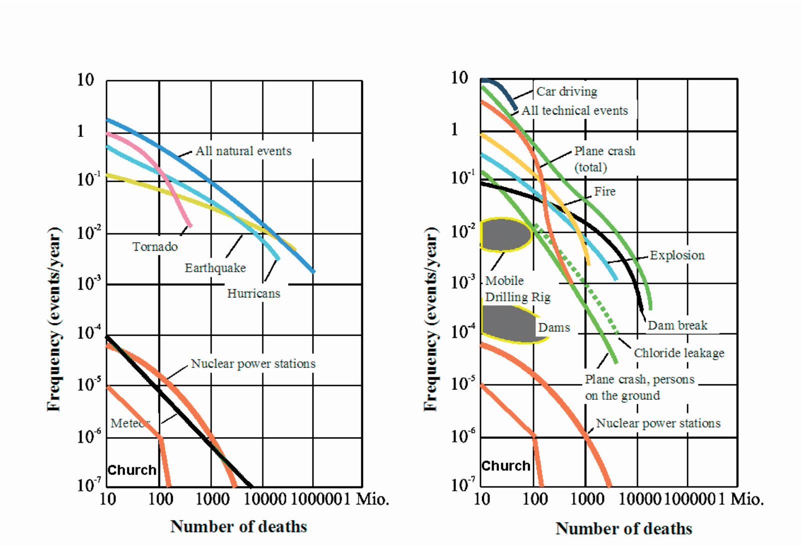


Figure 6 - 27: Natural and technical risks modified after [PROSKE 2002]

The results above and in figure 6-26 are based on different assumptions. While figure 6-26 is representative for single buildings, figure 6-27 provides information about accumulated events on a national or worldwide level. For comparison, the result for the single church is also plotted in figure 6-27. It can be found at the lower left edge of the each diagram. On a national level, the effects would be approximately three orders of magnitude higher. To assess risks on a national scale, it is necessary to include information about the number of buildings exposed to the hazard. This task and the derivation of acceptable risks will be explained in the following chapter.

6.8 Remarks on the acceptable risk

6.8.1 General considerations

Establishing criteria for acceptable risks is always a task followed by great discussion. It is never simple to create broadly acceptable borderlines. Thus – as it was already explained – in most applications two borderlines are used, dividing the risk into three regions. One part is called the region of acceptable risks, while the other is simply referred to as the unacceptable region. Between these two and of a size to be determined lies the ALARP region. The term *"as low as reasonably practicable"* already includes some vagueness in the evaluation of the size of this region.

Also, it has to be distinguished between individual risks, measured in mortality rates or FAR, and the societal risks expressed in the P-D diagrams. Some values, which might be used as background information, are the mortality rate of $9.0 \cdot 10^{-3}$ derived from table 2-8 or the acceptable probability of failure of $9.6 \cdot 10^{-6}$ evaluated using formula 2.3. Generally, comparison is based on a personally, locally or nationally acceptable level of risk. These terms were already introduced in chapter 2.3.6. The following text is concerned with the determination of the locally acceptable level of risk.

6.8.2 Minimum and maximum values

Minimum and maximum values are determined for national levels. In Europe, detailed approaches have been proposed by the HSE (Health and Safety Executive) in England [HSE 2001] and the VROM, i.e. the Dutch Ministry of Housing, Land Use Planning and Environment, in the Netherlands [JONKMAN ET AL. 2003]. Both vary especially in grading events with large consequences. In the application of formula 2-5, the HSE uses a value of one for n, while the VROM proposes to use a value of two. It cannot be the task of the author to propose national levels of acceptability, but for this application, the VROM rule is clearly favoured. In the opinion of the author, the society needs to value high consequences larger. This has to be done to account for secondary effects, such as breakdown of smaller economic systems and breakdown of community itself. Obviously, also the risk perception is higher for events causing a high number of casualties. To take these effects into account, figure 6-28 will compare both approaches by using the values listed in table 6-24.

Factor	Value
n	2 for VROM rule 1 for HSE
β	0.01 for broadly acceptable region 0.1 for unacceptable region
Nat_{size}	412 for Italy 576 for Germany
k	3
N_a	Depending on the country, Germany roughly 10000, Italy 50000 Only historical churches, if CSH values are included

Table 6 - 24: Data used in the evaluation of P-D diagrams shown in figure 6-28

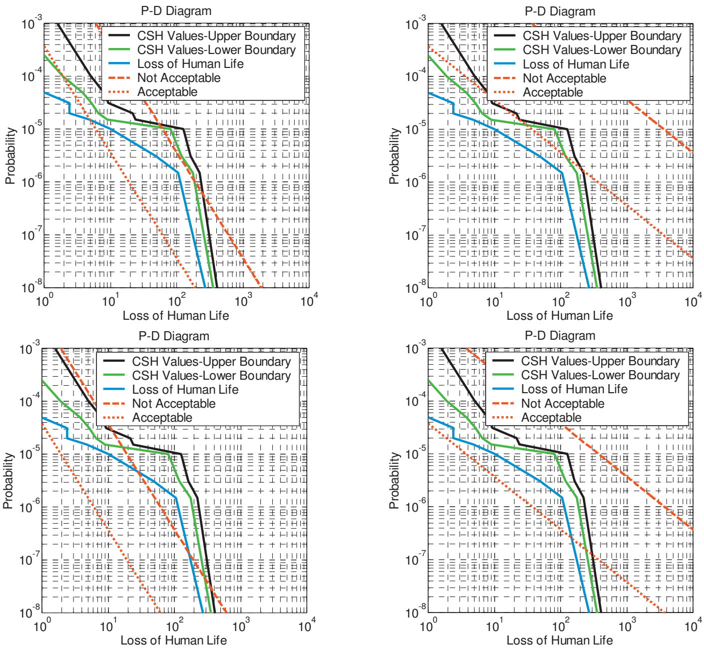


Figure 6 - 28: PD diagrams for the church if located in Germany (upper row) or Italy (lower row) using the VROM rule (left) and the HSE rule (right)

The picture above reveals very interesting information. Even if the church is located in Germany and the less strict, risk neutral criterion proposed by HSE is used, the church falls within the ALARP region – at least if the CSH values are included. The risk averse VROM criterion is not met in both cases. The results depend on the population size of the country and the number of affected buildings. The risk may also be compared to other measures explained in chapter 2 as it is shown in table 6-25.

Risk measure	Reference value	Calculated for the church	Acceptable
Probability of collapse	Formula 2.3: $9.6 \cdot 10^{-6}$	Page 168: 10^{-5}	No
Mortality rate	Table 2-16: $1 \cdot 10^{-4}$	Obtained by integrating all values in figure 6-26:roughly $1 \cdot 10^{-4}$, for 10000 churches in Germany with a population of 83 000 000 , resulting in $1.2 \cdot 10^{-7}$	Yes
FAR	Table 2-17: 3 to 60	Assuming that a person remains one hour in the church $1.2 \cdot 10^{-7} / 1 \cdot 10^{8} = 12.1$	ALARP

Table 6 - 25: Risk measures for the church

Risk measure	Reference value	Calculated for the church	Acceptable
Probability Damage diagram – local level	Figure 6-28	Figure 6-28	No/ALARP
Individual risk	Formula 2-8 with $\beta = 1 \cdot 10^{-4}$	Obtained by integrating all values in figure 6-26:$1 \cdot 10^{-4}$	No
National acceptable	Formula 2-6 and 2-12 with $\beta = 0.01$: 5.76 in Germany	10000 churches with a mean of 10 persons and a deviation of 3 persons inside with a probability of total collapse of 10^{-4}, applying formulas 2.10 and 2.11: 19	No
Proske optimal costs for safety measures	Reducing P_f from 10^{-4} to 10^{-5}	Using data from [PROSKE 2002] with 50 fatalities: 9330 €	

Table 6 -25 continued: Risk measures for the church

The output of the measures is quite different. While the mortality rate is the only value clearly fulfilling the requirements, all other values fall within the unacceptable region or at least in the ALARP zone. It must be admitted that the considered church is representing rather high values for humans inside the church. Although this is not representative for the majority of churches, even higher values could be obtained; either for facilities with a higher probability of failure, or those attracting an even larger number of tourists. So, if the national level is observed, more acceptable results would be obtained, because extreme cases are smoothed out. Nevertheless, on the local level even less favourable outcomes are possible.

A general recommendation cannot be given. What should become clear though, is that churches – even in Germany – may, depending on their attractiveness and structural response to ground motions, constitute risks that clearly fall within the unacceptable region. A full example is presented in chapter 7.

Chapter 7

Example application

7.1 Description of the building

This chapter aims at explaining how the results of this study may be used in practical applications. To be able to compare the results to other works, an example given in [AUGUSTI ET AL. 2001] or [AUGUSTI ET AL. 2000] will be used as a reference. The ground plan of the church and its dimension in metres are given in figure 7-1. If not shown otherwise, it is assumed that all walls have a thickness of 0.60 metres. The church was built in 1700.

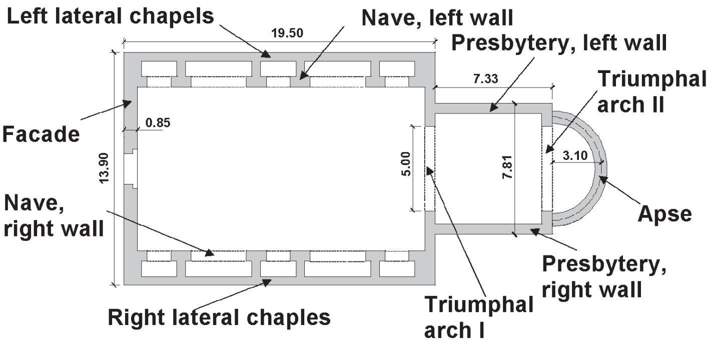


Figure 7- 1: Ground plan of the church

Main material parameters are given in table 7-1. Values indicated with an asterisk are taken from [AUGUSTI ET AL. 2000], other parameters are assumed.

Parameter	Distribution	Mean	Dev.	Min.	Max.
μ	lognormal	0.6	0.111	-	-
σ_{mr}*	lognormal	0.10 N/mm²	0.04	-	-
τ_{mr}*	lognormal	0.15 N/mm²	0.05	-	-
σ_{br}*	lognormal	2.0 N/mm²	0.5	-	-
τ_{br}	lognormal	1.0 N/mm²	0.3	-	-
η	lognormal	0.1	0.02	-	-
ρ*	normal	1800 kg/m³	150	-	-
E	normal	2000 N/mm²	240	-	-
β_m	triangular	0.8	-	0.3	1.0
β_b	triangular	0.4	-	0.3	1.0
c_{mt}	uniform	1.0	-	0.5	1.5
c_{bt}	uniform	1.0	-	0.5	1.5
adamp	uniform	0.52			
bdamp	uniform	0.0005			

Table 7 - 1: Material parameters and their distributions used in this chapter

No information is given about the distribution of visitors in the church. The density distribution of persons is thus evaluated with the program explained in chapter 6. The assumptions listed in table 7-2 were used as input data for the program. The church is seldom visited by tourists and only a medium number of persons attends religious events. This input results in the plots of figure 7-2. The size of the religious community is set to be 1250 persons for this example.

Input	Value		Input	Value
Number of inhabitants	0		Number of events / week	5
Number of workers	0		Number of events / weekend	2
Mean of visitors	5		Maximum number of persons	100
Standard deviation visitors	0.1		Standard deviation of maximum	20
Mean weekend	70		Number maximum is reached	3
Standard deviation weekend	20			
Mean week	30			
Standard deviation week	8			

Table 7 - 2: Assumptions for distribution of persons in the church

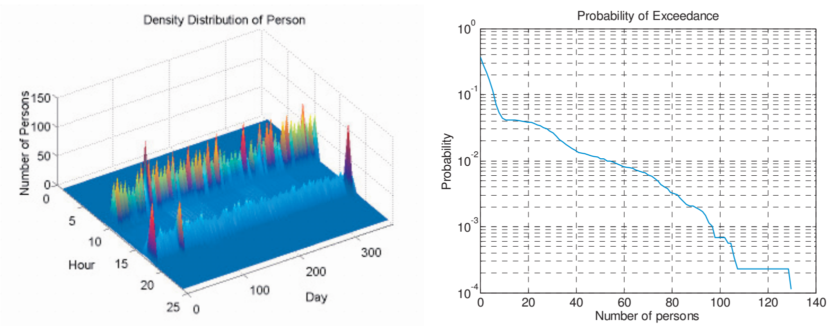


Figure 7- 2: Distribution of persons in the church

[AUGUSTI ET AL. 2000] do not explain if vaults are included in the structural layout, for their approach is not suitable to assess the vulnerability of vaults. Instead, they propose to represent the church by a composition of walls and larger macroelements as described in figure 7-3.

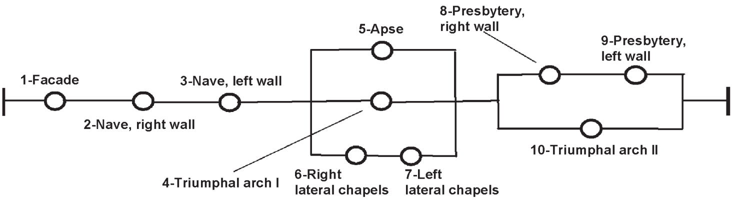


Figure 7- 3: Church logical diagram

The logical diagram also explains whether the elements construct a serial or a parallel system. By dividing the church into these elements and combining them in a logical diagram, the failure probabilities of each element are used to calculate the failure probability of the complete church. Therefore, depending on a parallel or serial system, formulas 7.1 for parallel systems or 7.2 for serial systems are used.

$$P_{f,par} = \prod_k P_k \tag{7.1}$$

$$P_{f,ser} = 1 - \prod_m (1 - P_m) \tag{7.2}$$

With P_k being the collapse probability of each macroelement k belonging to the parallel subsystem. P_m includes the collapse probability of each single macroelement or the collapse probability of each parallel subsystem. In contrast to figure 7-3, it is assumed that the church can be described as shown in figure 7-4. The altered structural configuration is taken to visualize, how vaults could be in included in the overall assessment.

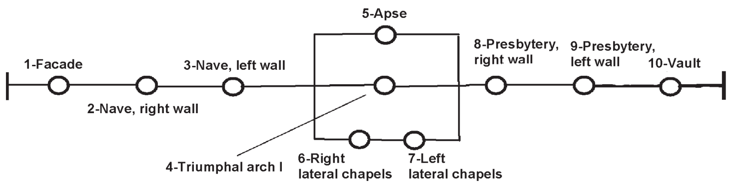


Figure 7- 4: Altered logical diagram of the church used in this chapter

7.2 Hazard assessment

For this application, the church is assumed to be located in an area which can be characterized by the Gutenberg-Richter coefficients of a = 3.0 and b = 0.95 in a radius of 50 km around the structure. The seismic hazard location is evenly distributed and the church is built on shallow soft soil. Using the program attached in appendix B, the output of figure 7-5 is obtained.

Figure 7-5a plots the yearly probability of exceedance for the magnitude using the Gutenberg-Richter relationship with the parameters previously explained. Figure 7-4b plots the resulting PGA values for different attenuation functions [AMBRASEYS ET AL. 1996], [SPUDICH ET AL. 1999], [ABRAHAMSON AND SILVA 1997] and [BOORE ET AL. 1997], figure 7-4c shows the yearly probability of exceedance for the PGV using the algorithm of [SABETTA AND PUGLIESE 1996] and finally, figure 7-4d plots the probability of duration using [SABETTA AND PUGLIESE 1996] in red and one proposed by [HERNANDEZ ET AL. 1999] in blue. The latter is taken as a representation of the significant duration D_{95} and the first as uniform duration $D_{u0.05g}$.

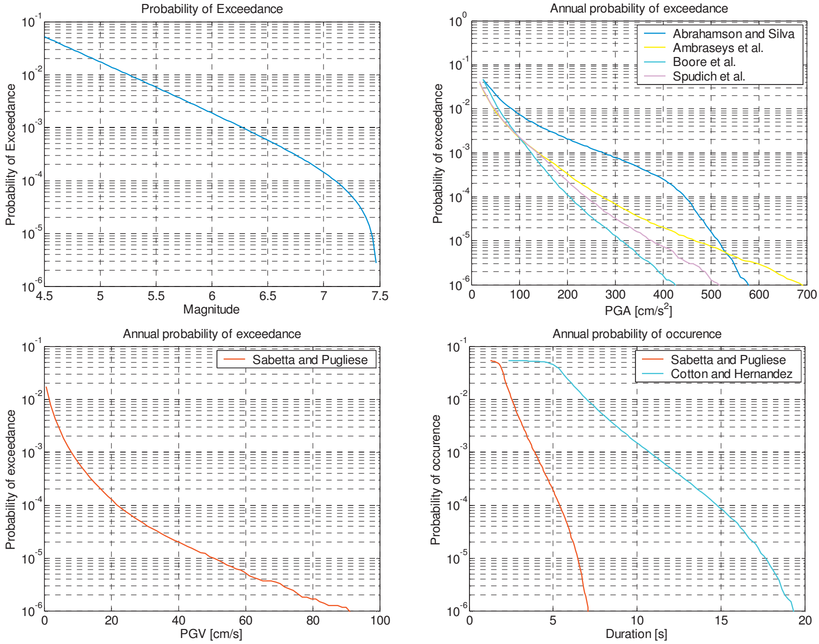


Figure 7- 5: Results of the hazard analysis for governing parameters, figure 7-4a upper left; figure 7-4b upper right; figure 7-4c lower left; figure 7-4d lower right;

7.3 Determination of vulnerability and damage

The probability of damage is then evaluated for discrete levels of ground motions. [AUGUSTI ET AL. 2000] considers a PGA of 0.16g and 0.4g. Thus, these two levels will also be evaluated in this chapter in order to provide comparable results. The approach is highlighted by two macroelements. At first, the lateral chapel wall will be analysed, followed by a computation of the barrel vault in the presbytery. For the other elements, the results are listed in table 7-7.

It is assumed that PGA, PGV and duration are directly correlated, thus for a PGA of 0.16g an annual probability of exceedance of 10^{-3} is predicted, if the algorithm of [SPUDICH ET AL. 1999] is used. A PGA of 0.4g corresponds to an annual probability of exceedance of $2{\cdot}10^{-5}$. Using these values the following data may be taken from figure 7-5.

Measure	Value for an annual probability of exceedance 10^{-3}	Value for an annual probability of exceedance $2{\cdot}10^{-5}$
PGA	160 cm/s²	400 cm/s²
PGV	9 cm/s	40 cm/s
$\mathbf{D_{S95}}$	10.8 s	17.0 s
$\mathbf{D_{U0.05g}}$	4.1 s	6.5 s

Table 7 - 3: Data for risk assessment

Now, an accelerogram is generated for a probability of exceedance of 10^{-3} with the intensity measures listed in table 7-3. This accelerogram is applied to the structure. Three calculations are performed, once for the worst combination of material parameters, once for the mean values of the material parameters and finally for the best combination. If this is done, plots as shown in figure 7-6 will result.

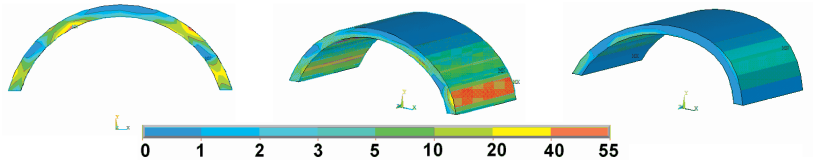


Figure 7- 6: Analysis of the vault, local mortar damage, left picture: worst combination of material parameters, middle picture: mean combination of material parameters, right picture: best combination of material parameters

The resulting values of the global mortar damage, suffered by the structure for each calculation, are given in table 7-4. In figure 4-39 and chapter 4.6.5.2 the damage was assessed to be exponentially distributed.

Best combination of material parameters	50
Mean combination of material parameters	170
Worst combination of material parameters	4500

Table 7 - 4: Results of the global mortar damage for each of the considered material combinations

Since the value of 50 is exceeded in all cases – corresponding to an exceedance probability of one – and the worst combination is assumed to occur with a probability of exceedance of 10^{-6}, the slope and location given by formula 3.31 may be determined with the following equations.

$$\log(1) = a - b \cdot 50 \tag{7.3}$$

$$\log(10^{-6}) = a - b \cdot 4500 \tag{7.4}$$

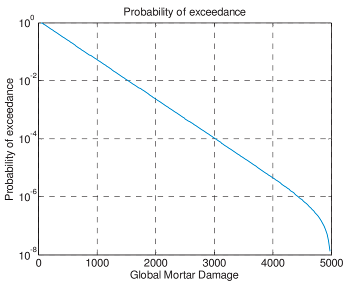


Figure 7- 7: Probability of exceedance for global mortar damage in the vault for a PGA of 0.16g

This results in values of a = 0.0674 and b = 0.0013. Using this, the probability of exceedance for this strong ground motion for the global mortar damage is given as plotted in following figure.

In table 5-2 the damage grades were already defined. As it can be seen in figure 7-6, a value of eight for the local mortar damage was calculated, if the best combination of material parameters is used. This value occurred in more than one location. The resulting damage grade is DG 2 according to table 5-2. Numerical collapse did not occur, thus DG 5 is not reached. In the worst case scenario, local damage did not exceed the value of 55. This value occurs at several places in the structure, the resulting damage grade is DG 3. Stating that DG 3 is reached, if the local mortar damage exceeds the value of 10, as proposed in table 5-2, then the probabilities of exceedance may be determined from figure 7-8. DG 2 is suffered in all cases while DG 3 is suffered with a probability of exceedance of 0.5. Multiplied with the probabilities of occurrence of the strong motion parameters, a probability of 10^{-3} for DG 2 and $5 \cdot 10^{-4}$ for DG 3 is calculated.

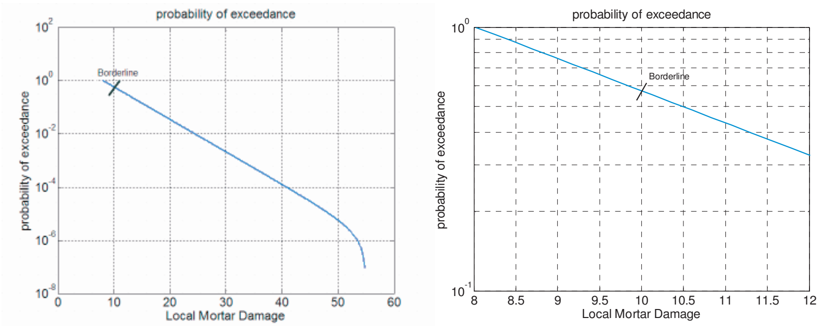


Figure 7- 8: Borderline between DG 2 and DG 3

The same procedure is used to predict damage probabilities for the right nave wall. The following results are obtained for the wall. As before, the brick damage was not as important as the mortar damage.

	Local mortar damage	Global mortar damage
Best combination of material parameters	25	1000
Mean combination of material parameters	5500	40 000
Worst combination of material parameters	5500*	120 000

* The wall failed numerically for the worst combination of material parameters, thus the results of the last computed load step are listed

Table 7 - 5: Results for the analysis of the wall

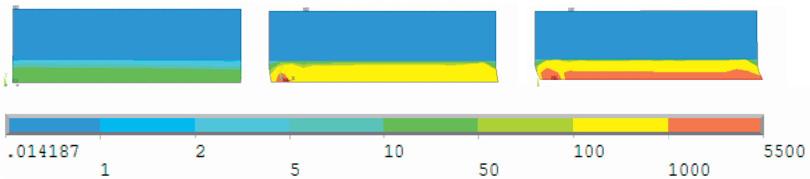


Figure 7- 9: Results for local mortar damage, analysis of the wall

The particularity of this analysis was that the wall failed for the worst combination of material parameters. Therefore, the last computed load step of the failed structure was taken as the reference for the maximum mortar damage parameters to occur. It is not possible to assign a probability to this size of mortar damage. Instead, a system of four equations with four unknowns has to be solved in order to determine the probability of each damage grade. The probability of exceedance for the best case scenario is again given as one. For the mean and worst case combination the probability is assigned a variable. The last equation is that the slope between the points generated by the best and mean combination has to be similar to the slope generated by the best and worst combination of material parameters.

If this system is solved, the following graph may be obtained, including the indication of damage grades. Respecting the probability of occurrence of the ground motion, a total failure probability of $2{\cdot}10^{-6}$ was computed.

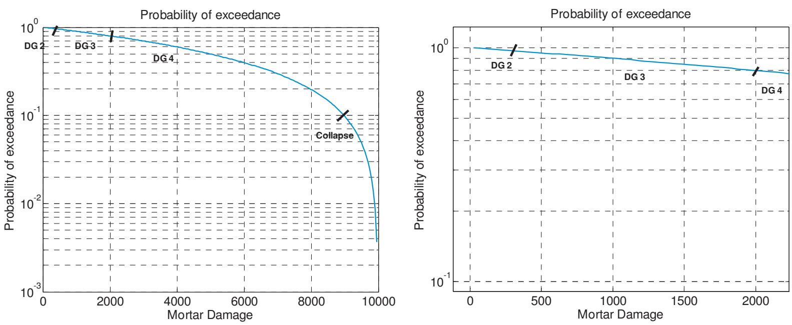


Figure 7- 10: Local mortar damage distribution and linked damage grades, right picture is a close-up of the left

Damage grade	Probability of exceedance for given ground motion	Total probability of exceedance
DG 1	1	$2{\cdot}10^{-5}$
DG 2	0.95	$1.9{\cdot}10^{-5}$
DG 3	0.80	$1.6{\cdot}10^{-5}$
DG 4	0.10	$2{\cdot}10^{-6}$

Table 7 - 6: Probabilities of damage grades for the considered wall

This has to be done for all macroelements and sufficient discrete levels of ground motions. The obtained results for all macroelements are listed in the following table.

Macroelement	**a_g=0.16g**				**a_g=0.40g**			
	DG1	DG2	DG3	DG4	DG1	DG2	DG3	DG4
Facade	10^{-3}	$7{\cdot}10^{-4}$	$3{\cdot}10^{-6}$	0	$2{\cdot}10^{-5}$	$2{\cdot}10^{-5}$	$1{\cdot}10^{-5}$	$9{\cdot}10^{-6}$
Right/left nave walls	10^{-3}	$6{\cdot}10^{-4}$	$2{\cdot}10^{-6}$	0	$2{\cdot}10^{-5}$	$2{\cdot}10^{-5}$	$1{\cdot}10^{-5}$	$8{\cdot}10^{-6}$
Triumphal arch between nave and presbytery	10^{-3}	$3{\cdot}10^{-4}$	0	0	$2{\cdot}10^{-5}$	$1{\cdot}10^{-5}$	$1{\cdot}10^{-5}$	$3{\cdot}10^{-6}$
Triumphal arch between presbytery and apse	10^{-3}	$2{\cdot}10^{-4}$	0	0	$2{\cdot}10^{-5}$	$1.5{\cdot}10^{-5}$	$1{\cdot}10^{-5}$	$4{\cdot}10^{-6}$
Right/left presb. wall	10^{-3}	$3{\cdot}10^{-4}$	0	0	$2{\cdot}10^{-5}$	$2{\cdot}10^{-5}$	$1{\cdot}10^{-5}$	$5{\cdot}10^{-6}$
Right/left chapel	10^{-3}	$5{\cdot}10^{-4}$	0	0	$2{\cdot}10^{-5}$	$1.5{\cdot}10^{-5}$	$1{\cdot}10^{-5}$	$4{\cdot}10^{-6}$
Apse	10^{-3}	$6{\cdot}10^{-4}$	$6{\cdot}10^{-6}$	0	$2{\cdot}10^{-5}$	$2{\cdot}10^{-5}$	$2{\cdot}10^{-5}$	$9{\cdot}10^{-6}$
Vault	10^{-3}	$5{\cdot}10^{-4}$	0	0	$2{\cdot}10^{-5}$	$2{\cdot}10^{-5}$	$2{\cdot}10^{-5}$	$8{\cdot}10^{-6}$

Table 7 - 7: Total damage probabilities for the church

7.4 Loss assessment

For the loss assessment, the importance factors are calculated in formulas 7.5 to 7.7. The mean values needed to calculate these values were taken from the sample study in the diocese of Aachen, Germany, which was explained in chapter 6. Additionally, the following assumptions apply.

- Human risk class 2, according to table 6-15 and 6-16
- Historical risk class 3, according to table 6-17
- Social risk class 3, according to table 6-18
- Cultural risk class 1, according to table 6-20, CIF assumed to be 0.3 since nearly no tourists visit the church

$$HIF = \frac{(1945 - 1700)}{(1945 - 1690)} \cdot e^{(1-\frac{1700}{1690})} = 1 \tag{7.5}$$

$$SIF = \left(30 + \frac{1250}{2221} \cdot 267 + \frac{30}{15} \cdot 267 \right) \cdot \frac{1}{673} = 1.06 \tag{7.6}$$

$$IF_{CSH} = 1.0 \cdot 0.3 + 1.0 \cdot 0.6 + 1.0 \cdot 1.0 = 1.9 \tag{7.7}$$

The points needed for the creation of the P-D diagram may now be determined as it was shown in table 6-22. After this is done, the following curves may be derived.

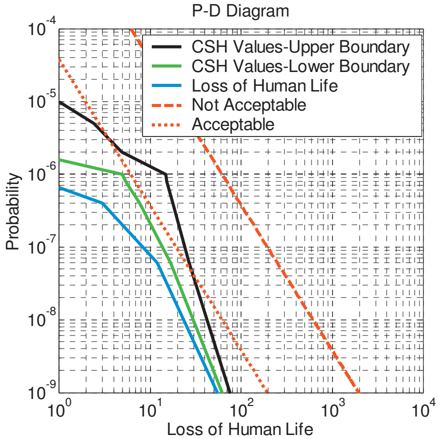


Figure 7- 11: Probability-Damage diagram for the church, using Italian parameters and the VROM rule, cf. table 6-24

Risk measure	Reference value	Calculated for the church	Acceptable
Probability of collapse	Formula 2.3: $9.6{\cdot}10^{-6}$	Table 7.7: $3{\cdot}10^{-5}$	Yes
Mortality rate	Table 2-16: $1{\cdot}10^{-4}$	Cf. table 6-25	Yes
FAR	Table 2-17: 3 to 60	Cf. table 6-25	ALARP
Probability Damage diagram – local level	Figure 7-11	Figure 7-11	Yes/ALARP
Individual risk	Formula 2-8 with β= 1: $1{\cdot}10^{-4}$	Obtained by integrating all values in figure 6-26: roughly $2{\cdot}10^{-5}$ for upper bound	Yes/ALARP
National acceptable	Formula 2-6 and 2-12 with β= 0.01: 56	Cf. table 6-25	ALARP

Table 7 - 8: Risk measures for the church

7.5 Comparison and summary

The full approach was shortly visualized. It is possible – with justifiable effort – to express the structural vulnerability and risk in probabilistic terms. The approach includes a first tentative proposal to compare CSH-values and include them into the risk assessment.

Regarding the results presented by [AUGUSTI ET AL. 2001] there is good agreement with the mean values and the prediction of damage probability for a given ground motion. What is different though, is the range of results for the different ground motion levels. [AUGUSTI ET AL. 2001] suppose normal distributions with a standard deviation, which is assumed by expert judgment based on the uncertainty in the material parameters. Consequently, they predict a rather wide range of possible damage grades for one special ground motion. Out of 7 different damage levels, which were also listed on page 124 of this work, they nearly always predict probabilities for each of the 7 damage levels to be unequal to zero. This means that even for high ground motions nearly no visible damages are predicted to a small percentage. The link to possible losses is not made, so further results cannot be compared.

Chapter 8

Synopsis

8.1 Summary

Risk assessment combines the probabilities of structural damage and its consequences. The goal of risk assessment is to compare diverse kinds of risks. Therefore, it is necessary to create measures and develop techniques which allow this comparison. Although most codes introduce very rough definitions for consequences to define target reliabilities, the detailed calculation is missing. Thus, risk assessment is a truly unique and innovative concept. It offers not only the ability to help decision makers in the description of risk priority classes. Moreover, it could influence the design of structures, if it would be developed further and introduced into building codes.

In this thesis, this concept was applied to historical masonry churches with respect to earthquake related damages. An extremely high number of uncertainties is faced, which have significant effects on the results of numerical simulations and damage predictions. Citing [ROTS 1997], it was believed that an accurate numerical approach is not possible, especially because of scattering material properties and the problems faced in the description of the masonry bond. To overcome these drawbacks, to enable the numerical description of masonry and to embed the results into the larger context of risk management, a very detailed procedure was followed.

In the first step, it was necessary to define large parts of the risk management process, since a great variety of terms is used in practice. This resulted in a greater ambiguity confusing both, analysts and decision makers. Also, a number of instruments were explained, which are used to evaluate and compare very different kinds of risks. The information given was rounded off by some general information on catastrophe occurrences and its comparison to health dependent mortality rates. It was necessary to include these causes and their effects in order to create awareness about how acceptable risk bases are derived and defined, and why and how those acceptable risks differ.

Afterwards, the influence of several uncertainties was assessed. At first, the simulations focused on the effects of ground motions. To determine those parameters, which are best correlated with the structural damage described by the material model used in this study, correlation coefficients for several damage parameters in dependence of the structural frequency were analysed. Interestingly, velocity related intensity measures proved to be best predictors of structural damage. Additionally, the influence of strong-motion duration was evaluated. The work offers information to describe the importance of duration independent of other parameters. The results were built-in and reflected in a program which provides results for diverse hazard scenarios including several attenuation functions for PGA, PGV, duration and Arias Intensity for single fault and uniform hazard areas, assuming either the Gutenberg-Richter relationship for the magnitude distribution or a Weibull distribution. Besides, natural accelerograms and several methods for the artificial computation of records were compared and their characteristics discussed.

The following task included the evaluation of the structural response to the ground motions. At first, the pushover method was compared to nonlinear dynamic calculation with the goal to explain, why the focus was set on nonlinear dynamic analyses. The sensitivity of 14 material and structural parameters was analysed in a first sample study. Afterwards, the results were verified by a large variety of test calculations. Distributions of the output parameters could be derived, which later were connected to discrete damage grades. The test calculations focused on elements which resembled macroelements, a concept used to divide the constructions into several substructures, which in turn have important effects on the overall structural reaction. The type of distributions for the outcome parameters proved to be the same no matter what element was tested. Similar results were obtained, even if different statistical moments for the probability distributions of the material parameters were used. Nevertheless, the importance of the input parameters differs for structural elements. Whereas for walls – as it would be guessed – the horizontal acceleration and the elastic modulus of the structure have the largest effect on the structural reaction, vaults reacted more sensible towards changes of the material properties, especially the shear strength of the mortar layers. The results obtained are not only of further use in this study; moreover they help to explain the structural behaviour in dynamic actions in a more accurate way than it was possible before. Results could also help to set priorities in experimental campaigns.

Before the risk could be evaluated eventually, the numerical results had to be connected to damage grades. Therefore, several existing measures were discussed. The definitions of the European Macroseismic Scale were found to be most appropriate and descriptions were derived, which connect the outcome of the numerical simulations by means of the two internal damage parameters to the five damage grades mentioned in the EMS.

Finally, possible consequences were evaluated by the creation of a database collecting typical information about churches. Information focused on human, economic and CSH consequences. For the prediction of economic losses and the creation of economic risk classes, sufficient information could not be collected. Nevertheless, the data provides a very good background to determine human and CSH losses. Risk classes were derived and connected to each other, so that a comparison with similar structures is possible. A tentative proposal was made to include CSH losses in the loss assessment by transferring them into Years of Life Lost. In this way, it is possible to compare the earthquake risk of historical structures to nearly all other possible risk types. The full approach was visualized for one example at the end.

8.2 Conclusions

Important new aspects of this work include the following:

- Definition of the risk management process and its components
- Evaluation of most important intensity parameters for tall masonry structures, with special focus on the effects of duration
- Evaluation of the sensitivity and probability distributions of input and output parameters, to assess their impact on the results of numerical simulations
- Connection of numerical results to discrete damage grades
- Assessment of possible consequences
- Enabling the comparison of risks

Although it was tried to include most uncertainties, their variety and the complex dynamic behaviour of masonry required some simplification and assumptions to keep the tasks feasible. The most important assumptions were that no soil-structure interaction was included in the study. Next, the material model is based on the assumption of plane stress. This enables the simulation to be performed in a reasonable amount of time, although it certainly is a simplification for highly stressed areas and vaults. The model was already applied to a greater number of structural models by the developers and proved to provide good to very good results, thus this drawback was accepted. Finally, throughout the simulations, acceleration, velocity and duration were assumed to be directly correlated. The scatter of duration for a given level of ground motion was not further analysed. The outline of the work was kept strictly modular, so that it will be possible to include new results, which are of relevance for this topic. This is why the author feels and sincerely hopes that the contribution made for the risk assessment is valuable information for both researchers and practitioners.

8.3 Outlook

During the work on this thesis some research topics occurred, which could not be dealt with. They shall be shortly mentioned to provide incentives for further research.

The approach itself could be enhanced by a more detailed investigation of churches in different nations. Also, the approach to include CSH values requires discussion and optimization if it would be used extensively to provide background for comparable risk analyses. Including the CSH in the LQI could be one of the developments derived from the data presented in this work.

Concerning the model and the numerical simulation, easier approaches should be developed to fasten the application. Throughout these efforts, various data have been presented which could be used for a more detailed description of pushover curves. Not only should pushover methods be switched from merely force-based to displacement-based methods. Including the results about the impact of duration could also improve the reliability of the data. Finally, the approach presented is only applicable with the applied material model. Only small additional research should be necessary to transfer results to simpler material models.

Appendix A

Risk management glossary

System:

The object of investigation for which all sources of hazard are identified and the risk analysis is being performed. The system can be composed by a single building or infrastructure element, a suburb of a city, a whole urban region or even an entire country.

Hazard:

A potentially adverse physical event, phenomenon or human activity that may cause harm to the predefined system. Harm can include injury or loss of life, property, cultural, social, historical and economic disruption or environmental degradation.

Hazard Analysis:

Consists of three steps: hazard identification, determination of relevant intensity levels and estimation of the corresponding probabilities of occurrence in a predefined time period. Depending on the size of the system, the results may differ for each element at risk.

Element at risk (EaR):

A single or a group of persons or objects within the predefined system that are susceptible and exposed to the impact of a hazard. In order to guarantee a complete coverage, all elements at risk collectively should compose the entire system that is being investigated. This will be referred to as the "principle of completeness".

Vulnerability: (for each EaR and hazard intensity)

Is a specific characteristic of an element at risk that indicates the susceptibility towards the impact of a hazard. Thus, vulnerability links the hazard intensity to the damage of an element at risk.

Damage: (for each EaR and hazard intensity)

Describes the physical, biological or chemical effect on an element at risk caused by the impact of a hazard of a given intensity. Damage captures the material harm and is not expressed in monetary terms.

Exposure:

Inventories of Elements at Risk that are subjected to a hazard.

Consequences: (for each EaR and hazard intensity)

This term captures and quantifies the various adverse effects an event of a certain intensity may have on the different elements at risk. Consequences can be subdivided into direct and indirect consequences. Direct as well as indirect consequences are to be further subdivided and classified into economic, humanitarian, ecological and CSH (cultural, social, historical) consequences due to the measure that is in use for their quantification. As it is possible to assign a monetary value only to economic consequences in a direct way, they will be referred to as tangible. All other classes of consequences are termed intangible.

Direct consequences:

Direct consequences are damages that occur simultaneously to the time the disaster takes place or by immediate follow-on physical destruction such as fires. Therefore they can directly be related to the disaster itself.

Economic: Adverse effects on capital stock resulting from physical damage of economic value carrying objects.

Humanitarian: Injuries and fatalities due to the damage of objects.

Ecological: Ground, air and water pollution, contamination of the environment or other devastating effects on ecosystems caused for instance by releases of toxic substances.

CSH: Adverse effects on capital stock resulting from physical damage of CSH value carrying objects.

Indirect consequences:

Indirect consequences in contrast usually occur with a time shift as a result of the direct consequences. They can be interpreted as follow up costs that result from the element at risk being not able to carry out its designated functionality within the system after the disaster has occurred.

Economic: Business interruption, wage losses, production downtime and other harms on the economy in the long term.

Humanitarian: The spread of diseases resulting from the absence of satisfactory hygiene within the affected area, psychological post-disaster effects.

Ecological: Penalties due to the violation of environmental regulation rules.

CSH: Adverse effects on the wellbeing of society resulting from the abandonment of the CSH value carrying object.

Loss: (for each consequence class and hazard intensity)

Subdivided by consequence class, this term accumulates all direct and indirect consequences a natural disaster of a certain intensity may have at the time the disaster occurs. To quantify the loss, the sum of all direct and discounted indirect consequences belonging to the considered consequence class for each element at risk being part of the system has to be calculated. In this connection the discounting of the indirect consequences is dependent on the time the consequences occur and the consequence class specific discount factor that is in use.

Then, by definition it can be distinguished between humanitarian, economic, ecological and CHS loss.

Structural Risk:

The structural risk can finally be calculated by taking the products of the annual probabilities of occurrence and the damages, both given as functions of the hazard intensity, and summing up these products over all hazard intensity levels.

= Probability x damage [damage measure / year]

Total Risk: (for each consequence class)

For each consequence class the risk can finally be calculated by taking the products of the annual probabilities of occurrence and the losses, both given as functions of the hazard intensity, and summing up these products over all hazard intensity levels.

Probability x loss [loss unit / year]

Consequently, the total risk is split into the humanitarian, the economic, the ecological and the CSH risk.

Appendix B

Program to evaluate the probability of earthquake intensities

```
% Code for MATLAB 6.5
% May be freely used on own risk
set(0,'defaultaxesfontsize',14);
set(0,'defaulttextfontsize',16);
set(0,'defaultlinelinewidth',2);

clear;
clc;
close all;
% __________________________________________________________________________
% __________________________________________________________________________
% ___________________________Input of parameters____________________________
% __________________________________________________________________________
% __________________________________________________________________________

% input of magnitudes
disp('Input of upper Magnitude (7.5)')
Mu=input('magnitude: Mu[-]=');
disp('Input of lower Magnitude (4.0)')
Mmin=input('magnitude: Mmin[-]=');

% input of Distribution for Magnitude
disp('Choose between Weibull for maxima (input=1) and Gutenberg-Richter (input=2)')
var1=input('Model=');

%input of seismic constants
if(var1==1)
disp('Input of seismic constant w')
w=input('constant: wmax[-]=');
disp('Input of seismic constant u')
u=input('constant: u[-]=');
disp('mean annual rate of occurence of these earthquakes')
mjue=input('constant: lambda[-]=');

elseif(var1==2)
disp('Input of seismic constant b')
b=input('constant: b[-]=')
disp('Input of seismic constant a')
a=input('constant: a[-]=');
mjue=exp(a*2.303-2.303*b*Mmin)

else
disp('Input error - please enter 1 oder 2')
end
```

```
% input of Distribution for Distance
disp('Choose between uniform hazard distribution (input=1) and givenfault (input=2)')
var2=input('Spatial distribution=');

if(var2==1)
%input of epicentral distance: Repi[km]
disp('Input of maximal epicentral distance (200km)')
Repimax=input('epicentral distance: Repimax[km]=');
disp('Input of minimum epicentral distance (0 km)')
Repimin=input('minimum epicentral distance: Repimin[km]=');

elseif(var2==2)
%fault parameters[km]
disp('Input of shortest distance to fault (20km)')
Repimina=input('shortest distance to fault[km]=');
disp('Input of fault length (100 km)')
lengthfault=input('fault length [km]=');
Repimin=Repimina+1     % otherwise numerical Instability because division by 0
Repimax=(Repimin^2+lengthfault^2/4)^.5;
else
disp('Input error - please enter 1 oder 2')
end

% input of geological conditions
disp('**********geological conditions**********')
disp('default: stiff soils')
disp('other conditions enter 1 for shallow')
disp('                      or 2 for deep')
Soil=input('                 Soil[1;2]=');
if isempty(Soil) == 1.0, Soil=0.0;end
S1=0.0;S2=0.0;
if Soil==1, S1=1.0;end
if Soil==2, S2=1.0;end

%input of simulation number
disp('Input of number of simulations to be performed')
disp('To insure consideration of low probabilities MCS > 10.000')
MCS=input('number of simulations: MCS[-]=');

%input of resolution
disp('frequency resolution for magnitude calculation/how many classes')
disp('suggestion: number of intervals= 1+3.3*log10(n)')
% 1926 Sturges scf Book Benjamin and Cornell
% Probability and Statistics for Engineers S. 8
num=input('number of classes =');

% ______________________________________________________________________________
% ______________________________________________________________________________
% _________________________Input of pdfs and cdfs________________________________
% ______________________________________________________________________________
% ______________________________________________________________________________

m=Mmin:(Mu-Mmin)/(num-1):Mu;
```

```
if(var1==1)
fm=(w./(Mu-u).*((Mu-m)./(Mu-u)).^(w-1).*exp(-((Mu-m)./(Mu-u)).^w))./(1-exp(-((Mu-Mmin)./(Mu-
u)).^w));

fmcum=cumsum(fm);
fmcummax=max(fmcum);
fmrel=fm/fmcummax;

z=0;
for n=1:1;
   z=z+1;
   q(:,z) = (Mu-Mmin)/n*fm';
   p=cumsum(q);
end

z=0;
N=max(p);
for n=1:1;
   z=z+1;
   q(:,z) = (Mu-Mmin)/n*fm';
   p1=cumsum(q)/N;
end

p2=1-p1;
c=max(fm);
FMber=fm/c;
FM=FMber';
clear FMber;
else

fm=exp(2.303.*a-2.303.*b*Mmin)*((exp(-2.303.*b*((m+0.01)-Mmin))-1.*exp(2.3.*-b.*(Mu-
Mmin)))/(1-1*exp(2.3*-b.*(Mu-Mmin))));
%fm=(2.303*b.*exp(-b*2.3.*((0.01+m)-Mmin)))./(1-exp(-b*2.3.*(Mu-Mmin)));
fmga=exp(2.303.*a-2.303.*b*Mmin)*((exp(-2.303.*b*((m+0.01)-Mmin))-1.*exp(2.3.*-b.*(Mu-
Mmin)))/(1-1*exp(2.3*-b.*(Mu-Mmin))));
%fmgab=exp(2.303.*a-2.303.*b*Mmin)*(exp(-2.303.*b*((m+0.01)-Mmin))-1.*exp(2.3.*-b.*(Mu-
Mmin)))/(1-1*exp(2.3*-b.*(Mu-Mmin)));

fmcum=cumsum(fm);
fmcummax=max(fmcum);
fmrel=fm/fmcummax;

z=0;
for n=1:1;
   z=z+1;
   q(:,z) = (Mu-Mmin)/n*fm';
   p=cumsum(q);
end

z=0;
N=max(p);
for n=1:1;
   z=z+1;
   q(:,z) = (Mu-Mmin)/n*fm';
   p1=cumsum(q)/N;
end
```

```
p2=1-p1;
c=max(fm);
FMber=fm/c;
FM=FMber';
end
clear FMber;

if(var2==1)
Repi=Repimin:(Repimax-Repimin)/(num-1):Repimax;
fr=2*Repi/(Repimax^2);
else
Repi=Repimin:(Repimax-Repimin)/(num-1):Repimax;
fr=2*Repi./(lengthfault*((Repi*1.001).^2-Repimin^2).^.5);
end

frcum=cumsum(fr);
frcummax=max(frcum);
frrel=fr/frcummax;

% ______________________________________________________________________________
% ______________________________________________________________________________
% ___________________________MCS PREPARATION_____________________________
% ______________________________________________________________________________
% ______________________________________________________________________________

%---------------------------------------------
%----------------Magnitudes----------------
%---------------------------------------------

% number of simulations per class
z=0;
for n=1:1;
   z=z+1;
   number(:,z) = MCS.*fmrel';
end
number2=round(number);


phi_anzahl=number2           ;% number of events per class
phi_out=Mmin:(Mu-Mmin)/num:Mu; % classifying
phi=linspace(Mmin,Mmin,MCS);   % generation of 1 x m matrix
phi=phi(1:MCS);

k=1;     % inititialize variable k
hammer = waitbar(0,'Please wait...1');
for j=1:num;
   for i=1:phi_anzahl(j);
      phi(k)=phi_out(j);     % generate phi for each class
      k=k+1; end
      waitbar(j/num)
   end
close(hammer)

% shuffling of vector phi to create statistical independence
dummy=phi(1:MCS);
dummy1=randperm(MCS);   % random generation of numbers (1:MCS)
```

```
dummy2=randperm(MCS);   % random generation of numbers (1:MCS)
hammer = waitbar(0,'Please wait...2');
for i=1:MCS;
   phi(dummy1(i))=dummy(dummy2(i));
   waitbar(i/length(phi));
end %shuffling all values
close(hammer)
phi=phi(1:MCS);

%---------------------------------------------
%----------------Distance-------------------
%---------------------------------------------

z=0;
for n=1:1;
   z=z+1;
   number3(:,z) = MCS.*frrel';
end

number4=round(number3);

phi_anzahla=number4;
phi_outa=Repimin:(Repimax-Repimin)/num:Repimax;
phia=linspace(Repimin,Repimin,MCS);

k=1;
hammer = waitbar(0,'Please wait...3');
for j=1:num;
   for i=1:phi_anzahla(j);
      phia(k)=phi_outa(j);
      k=k+1; end
   waitbar(j/num);
   end
close(hammer);

dummya=phia;
dummy1a=randperm(MCS);
dummy2a=randperm(MCS);
hammer = waitbar(0,'Please wait...4');
for i=1:MCS;
   phia(dummy1a(i))=dummya(dummy2a(i));
   waitbar(i/length(MCS));
end %shuffling all values
close(hammer);
phia=phia(1:MCS);

% ____________________________________________________________________________
% ____________________________________________________________________________
% ________________________Attenuation functions__________________________________
% ____________________________________________________________________________
% ____________________________________________________________________________
%---------------------------------------------
%------------Sabetta and Pugliese---------
%---------------------------------------------

%Arias intensity: IA[cm2/s3]
```

```
Ra=sqrt(phia.^2+5.3^2);
IA=exp(1.679+2.097.*phi-1.818*log(Ra)+0.561*S1+0.320*S2);

%Peak ground acceleration: Pga [cm/s2]
Rhp=sqrt(phia.^2+5.0^2);
Pga=981*10.^(-1.845+0.363*phi-log10(Rhp)+0.195*S1);

%regression for estimation of peak velocity: Pgv [cm/s]
Rvit=sqrt(phia.^2+3.9^2);
Iss=1.0;
if Soil==0.0, Iss=0.0;end
Pgv=10.^(-0.8281+0.489.*phi-log10(Rvit)+0.116*Iss);

%Duration of the ground motion strong phase: DV[s]
Rv=sqrt(phia.^2+5.1^2);
DV=exp(-1.802+0.445*phi+0.208*log(Rv)-0.307*S1+0.318*S2);

%---------------------------------------------
%------------Abrahamson and Silva------
%---------------------------------------------
%coefficients for evaluating f1
c1=6.4;
c4=5.6;
a1=1.64;
a2=0.512;
a12=0;
a3=-1.1450;
a13=0.17;
n=2;
a4=-0.144;
%coefficients for evaluating f3
a5=0.61;
a6=0.26;

%coefficients for evaluating f4
a9=0.37;

%coefficients for evaluating f5
a10=-0.42;
a11=-0.23;
c5=0.03;
F=0; % Parameter for fault type, could be changed
HW=1; % Parameter for hanging wall type, could be changed

%determination of f1
if(phi<=c1)
f1=a1+a2.*(phi-c1)+a12.*(8.5-phi).^n+(a3+a13.*(phi-c1)).*log((phia.^2.+c4^2).^0.5);
else
f1=a1+a4.*(phi-c1)+a12.*(8.5-phi).^n+(a3+a13.*(phi-c1)).*log((phia.^2.+c4^2).^0.5);
end

%determination of f3
if(phi<=5.8)
f3=a5;
elseif(5.8<phi&phi<c1)
```

```
f3=a5+(a6-a5)/(c1-5.8);
else
f3=a6;
end

%determination of f4
if(phi<=5.5)
fHWM=0;
elseif(5.5<phi&phi<6.5)
fHWM=phi-5.5;
else
fHWM=1;
end

if(phia<=4)
fHWR=0;
elseif(4<phia&phia<=8)
fHWR=a9*(phia-4)/4;
elseif(8<phia&phia<=18)
fHWR=a9;
elseif(18<phia&phia<=24)
fHWR=a9*(1-((phia-18)/7));
else
fHWR=0;
end

f4=fHWM.*fHWR;

%determination of f5
pgarock=exp(f1+F*f3+HW*f4);

f5=a10+a11*log(pgarock+c5);

if isempty(Soil) == 1.0, Soil=0.0;end
if Soil==1, S=0.5;end
if Soil==2, S=1.0;end

Att_AS=1.*exp(f1+F.*f3+HW.*f4+S.*f5);

%----------------------------------------------
%-------------Ambraseys et. al-------------
%----------------------------------------------

%coefficients for attenuation Ambraseys et. al
Caea1=-1.48;
Caea2=0.266;
Caea4=-0.922;
h0=3.5;
Ca=0.117;
Cs=0.124;

if isempty(Soil) == 1.0, Soil=0.0;end
Sa=0.0;Ss=0.0;
if Soil==1, Sa=1.0;Ss=0.0;end
if Soil==2, Sa=0.0;Ss=1.0;end
```

```
Att_Aea=10.^(Caea1+Caea2.*phi+Caea4.*log10((phia.^2.+h0^2).^0.5)+Ca*Sa+Cs*Ss);

%---------------------------------------------
%--Sabetta and Pugliese- Cross-Check--
%---------------------------------------------

%coefficients for attenuation Sabetta Pugliese as a test

if isempty(Soil) == 1.0, Soil=0.0;end
SI=0;
if Soil==1, SI=0.0;end
if Soil==2, SI=1.0;end

Att_SPCC=981.*10.^(0.306.*phi-log10((phia.^2+5.8^2).^0.5)+0.169*SI-1.56); %mit log oder
log10???

%---------------------------------------------
%--------------Boore Joyner Fumal-------
%---------------------------------------------

%coefficients for attenuation Boore Joyner Fumal
b1=-0.242; %rupture mechanism not specified
b2=0.527;
b3=0;
b5=-0.778;
bv=-0.371;
h=5.57;
Va=1396;

if isempty(Soil) == 1.0, Soil=0.0;end
Vs=800;
if Soil==1, Vs=500;end
if Soil==2, Vs=310;end

Att_JBF=1.*exp(b1+b2.*(phi-6)+b3.*(phi-6).^2+b5.*log((phia.^2+h^2).^0.5)+bv*(log(Vs/Va)));

%---------------------------------------------
%--------------------SEA 99----------------
%---------------------------------------------

%coefficients for attenuation SEA 99
b1SEA=0.237;
b2SEA=0.229;
b3SEA=0;
b5SEA=-1.052;
b6SEA=0.174;
hSEA=5.57;

if isempty(Soil) == 1.0, Soil=0.0;end
lambda=0;
if Soil==1, lambda=0.8;end
if Soil==2, lambda=1.0;end

Att_SEA=10.^(b1SEA+b2SEA.*(phi-6)+b3SEA.*((phi-
6).^2)+b5SEA.*log10((phia.^2.+hSEA^2).^0.5)+b6SEA*lambda);
```

```
%______________________________________________
%---------------------------------------------------------------------
%--Zonno Montaldo attenuation Arias Intensity--------------
%---------------------------------------------------------------------
%______________________________________________

if isempty(Soil) == 1.0, Soil=0.0;end
AI_ZM=10.^(0.713+0.664.*phi-1.046.*log10(phi));
if Soil==1, AI_ZM=10.^(0.713+0.664.*phi-1.046.*log10(phia)+0.075*1);end%log oder log10
if Soil==2, AI_ZM=10.^(0.713+0.664.*phi-1.046.*log10(phia)+0.075*1);end%log oder log10

%______________________________________________
%---------------------------------------------------------------------
%--Travasarou, Bray Abrahamson attenuation Arias Intensity--
%-------------------------------------------------------------------
%______________________________________________

FN=0;
FR=0;
if isempty(Soil) == 1.0,Sc=0,Sd=0;end

if Soil==1, Sc=1,Sd=0;end%log oder log10
if Soil==2, Sc=0,Sd=1;end%log oder log10
AI_TBA=1/9810.*exp(2.8-1.981.*(phi-6)+20.72.*(phi/6)-
1.703.*log((phia.^2+8.78^2).^.5)+(0.454+0.101.*(phi-6))*Sc+(0.479+0.334.*(phi-6))*Sd-
0.166*FN+0.512*FR);

%______________________________________________
%---------------------------------------------------------------------
%----Duration Olafsson, Remseth, Sigbjörnson--------------
%---------------------------------------------------------------------
%______________________________________________

%coefficients for attenuation
theta1=-4.727;
theta2=0.0517;
M0=1000000*10.^(1.5.*phi+10.7); % Factor to transfer units
theta3=0.663;

Dur_OLAF=1.*exp(theta1+theta2.*log(M0)+theta3.*log(phia));%log oder 10

%______________________________________________
%---------------------------------------------------------------------
%-------Duration Midorikawa and Kobayashi------------------
%---------------------------------------------------------------------

Dur_MK=0.013*10.^(0.42.*phi)+0.24.*phia;

%______________________________________________
%---------------------------------------------------------------------
%-------Duration  and Cotton Hernandez------------------------
%---------------------------------------------------------------------

Dur_HC=1.*exp(-1.04+0.44.*phi+0.19.*log(phia)+0.04*1);
```

```
% ______________________________________________________________________________
% ______________________________________________________________________________
% __________________________________Plots_______________________________________
% ______________________________________________________________________________
% ______________________________________________________________________________

Att_ASa = Att_AS*981;
Att_Aeaa = Att_Aea*981;
Att_SPCCa = Att_SPCC;
Att_JBFa = Att_JBF*981;
Att_SEAa = Att_SEA*981;

scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(1)
subplot(1,3,1)
hist(IA,num)
title('Histogram Arias Intensity')
xlabel('Arias Intensity');
ylabel('Absolute frequency');
subplot(1,3,2)
hist(AI_ZM,num)
title('Histogram Arias Intensity Zonno Montaldo')
xlabel('PGA');
ylabel('Absolute frequency');
subplot(1,3,3)
hist(AI_TBA,num)
title('Histogram Arias Traaousr')
xlabel('PGA');
ylabel('Absolute frequency');

set(gca,'fontsize',16); %axis
scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
figure(2)
subplot(2,3,1)
hist(Pga,num)
title('Histogram PGA')
xlabel('PGA');
ylabel('Absolute frequency');
subplot(2,3,2)
hist(Att_ASa,num)
title('Histogram PGA Abrahamson and Silva')
xlabel('PGA');
ylabel('Absolute frequency');
subplot(2,3,3)
hist(Att_Aeaa,num)
title('Histogram PGA Ambraseys et. al')
xlabel('PGA');
ylabel('Absolute frequency');

subplot(2,3,4)
hist(Att_SPCCa,num)
title('Histogram PGA Sabetta Pugliese check')
xlabel('PGA');
```

```
ylabel('Absolute frequency');

subplot(2,3,5)
hist(Att_JBFa,num)
title('Histogram PGA Boore Joyner Fumal')
xlabel('PGA');
ylabel('Absolute frequency');

subplot(2,3,6)
hist(Att_SEAa,num)
title('Histogram SEA 99')
xlabel('PGA');
ylabel('Absolute frequency');

%-----------------------------------------------------

scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(3)
subplot(2,2,1)
hist(DV,num)
title('Histogram Time Duration')
xlabel('Time');
ylabel('Absolute frequency');
subplot(2,2,2)
hist(Dur_OLAF,num)
title('Histogram Time Duration Olaffson')
xlabel('Time');
ylabel('Absolute frequency');
subplot(2,2,3)
hist(Dur_MK,num)
title('Histogram Time Duration Midorikawa')
xlabel('Time');
ylabel('Absolute frequency');;
subplot(2,2,4)
hist(Dur_HC,num)
title('Histogram Time Hernandez Cotton')
xlabel('Time');
ylabel('Absolute frequency');;

%-----------------------------------------------------
[aPga,asnum]=hist(Pga,num);
[aAtt_AS,anum]=hist(Att_ASa,num);
[aAtt_Aea,bnum]=hist(Att_Aeaa,num);
[aAtt_SPCC,cnum]=hist(Att_SPCC,num);
[aAtt_JBF,dnum]=hist(Att_JBFa,num);
[aAtt_SEA,enum]=hist(Att_SEAa,num);

[aIA,aIAnum]=hist(IA,num);
[aZM,bIAnum]=hist(AI_ZM,num);
[aTBA,cIAnum]=hist(AI_TBA,num);

[aDV,adnum]=hist(DV,num);
[aOLAF,bdnum]=hist(Dur_OLAF,num);
[aMK,cdnum]=hist(Dur_MK,num);
```

```
[aHC,ddnum]=hist(Dur_HC,num);

[aPgv,ednum]=hist(Pgv,num);

%-----------------------------------------------------

maxpga=max(Pga);
minpga=min(Pga);
maxpgaAtt_AS=max(Att_ASa);
minpgaAtt_AS=min(Att_ASa);
maxpgaAtt_Aea=max(Att_Aeaa);
minpgaAtt_Aea=min(Att_Aeaa);
maxpgaAtt_JBF=max(Att_JBFa);
minpgaAtt_JBF=min(Att_JBFa);
maxpgaAtt_SEA=max(Att_SEAa);
minpgaAtt_SEA=min(Att_SEAa);

maxpgaIA=max(IA);
minpgaIA=min(IA);
maxpgaAI_ZM=max(AI_ZM);
minpgaAI_ZM=min(AI_ZM);
maxpgaAI_TBA=max(AI_TBA);
minpgaAI_TBA=min(AI_TBA);

maxpgaDV=max(DV);
minpgaDV=min(DV);
maxpgaOLAF=max(Dur_OLAF);
minpgaOLAF=min(Dur_OLAF);
maxpgaMK=max(Dur_MK);
minpgaMK=min(Dur_MK);
maxpgaHC=max(Dur_HC);
minpgaHC=min(Dur_HC);

maxpgv=max(Pgv);
minpgv=min(Pgv);


numa=minpga:(maxpga-minpga)/(num-1):maxpga;
numb=minpgaAtt_AS:(maxpgaAtt_AS-minpgaAtt_AS)/(num-1):maxpgaAtt_AS;
numc=minpgaAtt_Aea:(maxpgaAtt_Aea-minpgaAtt_Aea)/(num-1):maxpgaAtt_Aea;
numd=minpgaAtt_JBF:(maxpgaAtt_JBF-minpgaAtt_JBF)/(num-1):maxpgaAtt_JBF;
nume=minpgaAtt_SEA:(maxpgaAtt_SEA-minpgaAtt_SEA)/(num-1):maxpgaAtt_SEA;

numf=minpgaIA:(maxpgaIA-minpgaIA)/(num-1):maxpgaIA;
numg=minpgaAI_ZM:(maxpgaAI_ZM-minpgaAI_ZM)/(num-1):maxpgaAI_ZM;
numh=minpgaAI_TBA:(maxpgaAI_TBA-minpgaAI_TBA)/(num-1):maxpgaAI_TBA;

numi=minpgaDV:(maxpgaDV-minpgaDV)/(num-1):maxpgaDV;
numj=minpgaOLAF:(maxpgaOLAF-minpgaOLAF)/(num-1):maxpgaOLAF;
numk=minpgaMK:(maxpgaMK-minpgaMK)/(num-1):maxpgaMK;
numl=minpgaHC:(maxpgaHC-minpgaHC)/(num-1):maxpgaHC;

numav=minpgv:(maxpgv-minpgv)/(num-1):maxpgv;

testa=aPga/MCS*mjue;
testb=aAtt_AS/MCS*mjue;
```

```
testc=aAtt_Aea/MCS*mjue;
testd=aAtt_JBF/MCS*mjue;
teste=aAtt_SEA/MCS*mjue;

testf=aIA/MCS*mjue;
testg=aZM/MCS*mjue;
testh=aTBA/MCS*mjue;

testi=aDV/MCS*mjue;
testj=aOLAF/MCS*mjue;
testk=aMK/MCS*mjue;
testl=aHC/MCS*mjue;

testav=aPgv/MCS*mjue;

%__________________________________________________
%__________________________________________________
%__________________________________________________


ptesta=cumsum(testa);
ptestb=cumsum(testb);
ptestc=cumsum(testc);
ptestd=cumsum(testd);
pteste=cumsum(teste);
ptestf=cumsum(testf);
ptestg=cumsum(testg);
ptesth=cumsum(testh);
ptesti=cumsum(testi);
ptestj=cumsum(testj);
ptestk=cumsum(testk);
ptestl=cumsum(testl);
ptestav=cumsum(testav);

nb=max(ptesta);
nba=max(pteste);
nbb=max(ptestj);
nbv=max(ptestav);
paba=1/nb.*ptesta;

pbtesta=nb-ptesta;
pbtestb=nb-ptestb;
pbtestc=nb-ptestc;
pbtestd=nb-ptestd;
pbteste=nb-pteste;
pbtestf=nba-ptestf;
pbtestg=nba-ptestg;
pbtesth=nba-ptesth;
pbtesti=nbb-ptesti;
pbtestj=nbb-ptestf;
pbtestk=nbb-ptestk;
pbtestl=nbb-ptestl;
pbtestav=nbv-ptestav;
%__________________________________________________
%__________________________________________________
%__________________________________________________
```

```
scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(4);
plot(asnum,aPga,'r-')
hold on
plot(anum,aAtt_AS,'b-')
hold on
plot(bnum,aAtt_Aea,'y-')
hold on
plot(cnum,aAtt_JBF,'c-')
hold on
plot(dnum,aAtt_SEA,'m-')
hold off
title('Histogram PGA ')
xlabel('PGA [cm/s^2]');
ylabel('Absolute frequency');
h = legend('Sabetta and Pugliese','Abrahamson and Silva','Ambraseys et al.','Boore et al.','Spudich et
al.',1);

scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(5);
plot(numa,testa,'r');
hold on
plot(numb,testb,'b');
hold on
plot(numc,testc,'y');
hold on
plot(numd,testd,'c');
hold on
plot(nume,teste,'m');
hold off
title('Annual probability of occurence')
xlabel('PGA [cm/s^2)');
ylabel('Probability of occurence');
h = legend('Sabetta and Pugliese','Abrahamson and Silva','Ambraseys et al.','Boore et al.','Spudich et
al.',1);

scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(6);
plot(numf,testf,'r');
hold on
plot(numg,testg,'b');
hold on
plot(numh,testh,'y');
hold off
title('Annual probability of occurence')
xlabel('Arias Intensity');
ylabel('Probability of occurence');
h = legend('Sabetta and Pugliese','Zonno and Montaldo','Travasarou, Bray, Abrahamson.',1);
```

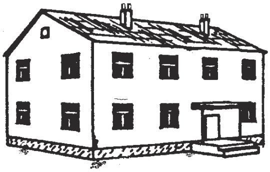
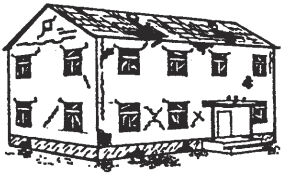
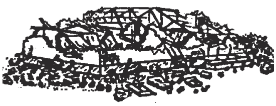
```
scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(7);
plot(numi,testi,'r');
hold on
plot(numj,testj,'b');
hold on
plot(numk,testk,'y');
hold on
plot(numl,testl,'c');
hold off
title('Annual probability of occurence')
xlabel('Duration [s]');
ylabel('Probability of occurence');
h = legend('Sabetta and Pugliese','Olafsson, Remseth, Sigbjörnson','Duration Midorikawa and
Kobayashi','Cotton and Hernandez',1);

%----------------------------------------------

clear z
scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(8);
semilogy(numa,pbtesta,'r');
hold on
semilogy(numb,pbtestb,'b');
hold on
semilogy(numc,pbtestc,'y');
hold on
semilogy(numd,pbtestd,'c');
hold on
semilogy(nume,pbteste,'m');
hold off
title('Annual probability of exceedance')
xlabel('PGA [cm/s^2]');
ylabel('Probability of exceedance');
h = legend('Sabetta and Pugliese','Abrahamson and Silva','Ambraseys et al.','Boore et al.','Spudich et
al.',1);

scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(9);
semilogy(numf,pbtestf,'r');
hold on
semilogy(numg,pbtestg,'b');
hold on
semilogy(numh,pbtesth,'y');
hold off
title('Annual probability of exceedance')
xlabel('Arias Intensity');
ylabel('Probability of exceedance');
h = legend('Sabetta and Pugliese','Zonno and Montaldo','Travasarou, Bray, Abrahamson.',1);
```

```
scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(10);
plot(numi,pbtesti,'r');
hold on
plot(numj,pbtestj,'b');
hold on
plot(numk,pbtestk,'y');
hold on
plot(numl,pbtestl,'c');
hold off
title('Annual probability of occurence')
xlabel('Duration [s]');
ylabel('Probability of occurence');
h = legend('Sabetta and Pugliese','Olafsson, Remseth, Sigbjörnson','Duration Midorikawa and
Kobayashi','Cotton and Hernandez',1);

scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(11);
semilogy(numi,pbtesti,'r');
hold on
semilogy(numk,pbtestk,'y');
hold on
semilogy(numl,pbtestl,'c');
hold off
title('Annual probability of occurence')
xlabel('Duration [s]');
ylabel('Probability of occurence');
h = legend('Sabetta and Pugliese','Duration Midorikawa and Kobayashi','Cotton and Hernandez',1);
grid on
axis([1 12 0.000001 0.01])

figure(12)
semilogy(m,fmga,'b');
title('Probability of exceedance')
xlabel('Magnitude');
ylabel('probability of exceedance');
title('Annual probability of occurence')
xlabel('Duration [s]');
ylabel('Probability of occurence');

scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(13);
semilogy(numav,pbtestav,'r');
title('Annual probability of exceedance')
xlabel('PGV [cm/s]');
ylabel('Probability of exceedance');
h = legend('Sabetta and Pugliese',1);
```

Appendix C

The short form of EMS-98 [GRÜNTHAL 1998]

EMS intensity	Definition	Description of typical observed effects (abstracted)
I	**Not felt**	Not felt.
II	**Scarcely felt**	Felt only by very few individual people at rest in houses.
III	**Weak**	Felt indoors by a few people. People at rest feel a swaying or light trembling.
IV	**Largely observed**	Felt indoors by many people, outdoors by very few. A few people are awakened. Windows, doors and dishes rattle.
V	**Strong**	Felt indoors by most, outdoors by few. Many sleeping people awake. A few are frightened. Buildings tremble throughout. Hanging objects swing considerably. Small objects are shifted. Doors and windows swing open or shut.
VI	**Slightly damaging**	Many people are frightened and run outdoors. Some objects fall. Many houses suffer slight non-structural damage like hair-line cracks and fall of small pieces of plaster.
VII	**Damaging**	Most people are frightened and run outdoors. Furniture is shifted and objects fall from shelves in large numbers. Many well built ordinary buildings suffer moderate damage: small cracks in walls, fall of plaster, parts of chimneys fall down; older buildings may show large cracks in walls and failure of fill-in walls.
VIII	**Heavily damaging**	Many people find it difficult to stand. Many houses have large cracks in walls. A few well built ordinary buildings show serious failure of walls, while weak older structures may collapse.
IX	**Destructive**	General panic. Many weak constructions collapse. Even well built ordinary buildings show very heavy damage: serious failure of walls and partial structural failure.
X	**Very destructive**	Many ordinary well built buildings collapse.
XI	**Devastating**	Most ordinary well built buildings collapse; even some with good earthquake resistant design are destroyed.
XII	**Completely devastating**	Almost all buildings are destroyed.

Classification of damage to masonry buildings	
	**Grade 1: Negligible to slight damage (no structural damage, slight non-structural damage)** Hair-line cracks in very few walls. Fall of small pieces of plaster only. Fall of loose stones from upper parts of buildings in very few cases.
	**Grade 2: Moderate damage (slight structural damage, moderate non-structural damage)** Cracks in many walls. Fall of fairly large pieces of plaster. Partial collapse of chimneys.
	**Grade 3: Substantial to heavy damage (moderate structural damage, heavy non-structural damage)** Large and extensive cracks in most walls. Roof tiles detach. Chimneys fracture at the roof line; failure of individual non-structural elements (partitions, gable walls).
	**Grade 4: Very heavy damage (heavy structural damage, very heavy non-structural damage)** Serious failure of walls; partial structural failure of roofs and floors.
	**Grade 5: Destruction (very heavy structural damage)** Total or near total collapse.

Appendix D

Generation of artificial earthquake after Sabetta and Pugliese [SABETTA AND PUGLIESE 1996].

The lognormal function used to determine the physical spectrum is given by the following formula and parameters:

$$PS_{approx}(t,f) = \frac{Pa(t)}{f \cdot \sqrt{2\pi\delta}} \cdot e^{[-\ln f - \ln\beta(t)]^2 / 2\delta^2} \tag{D-1}$$

$$\ln\beta(t) = \ln Fc(t) - \delta^2 / 2 \tag{D-2}$$

$$\delta = \sqrt{\ln[1 + Fb^2(t) / Fc^2(t)} \tag{D-3}$$

$$\ln(Fc(T)) = 3.4 - 0.35 \cdot \ln(t) - 0.218 \cdot M - 0.15 \cdot S_2 \tag{D-4}$$

$$Fb = (0.44 - 0.05 \cdot M - 0.08 \cdot S_1 + 0.03 \cdot S_2) \cdot Fc \tag{D-5}$$

$$Pa(t) = \frac{AI}{t \cdot \sqrt{2\pi\sigma}} \cdot e^{-[\ln(t) - \mu]^2 / 2\sigma^2} \tag{D-6}$$

$$\mu = \ln(T2) + \sigma^2 \tag{D-7}$$

$$\sigma = \ln(T3 / T2) / 2.5 \tag{D-8}$$

T1=R/7; T2=T1+0.5DV; T3=T1+2.5*DV; Total duration=1.3*T3

Where M is the Magnitude (M_s, when M_s and M_L are greater or equal than 5.5 otherwise M_L). f the frequency. Fc(t) is in Hz units, t is time in seconds, S1 and S2 are parameters for describing the soil type; R is the epicentral distance and DV the total duration of the strong motion phase

The artificial accelerogram may now be generated by the summation:

$$a(t) = 2 \cdot \sum C_n(t) \cos(n 2\pi f_0 t + \varphi_n) \quad \text{(D-9)}$$

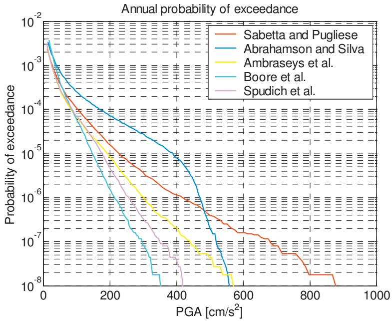
$$C_n(t) = \sqrt{2\pi f_0 PS(f_n, t)} \quad \text{(D-10)}$$

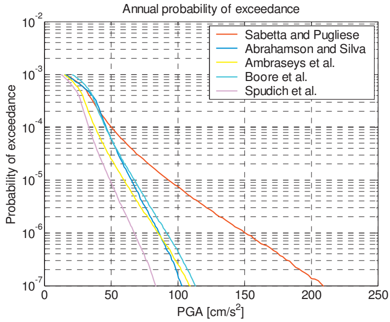
Where a(t) is the, f_0 the fundamental frequency, i.e. the reciprocal of the total duration and the phases are uniformly distributed random numbers in the range from 0 to 2p.

Appendix E

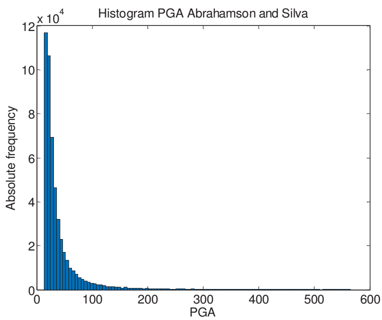
Sample results of hazard analyses

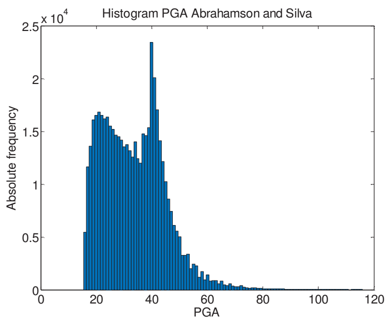
PGA stiff soil, Gutenberg Richter relationship, b = 1.3, a = 3.5,
PGA probability of exceedance,
uniform hazard, radius 75 km (left) – fault 40 km shortest distance (right)



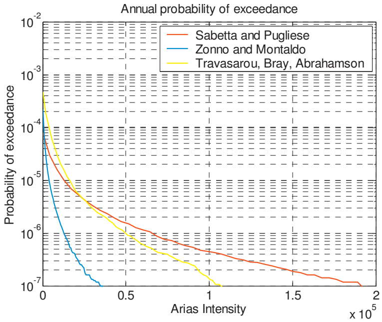


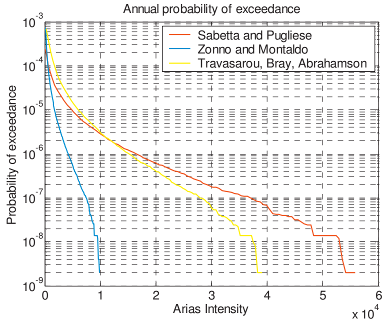
PGA stiff soil, Gutenberg Richter relationship, b = 1.3, a = 3.5,
PGA histograms for 500.000 simulations,
uniform hazard, radius 75 km (left) – fault 40 km shortest distance (right)



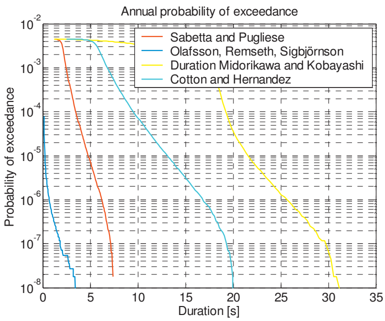


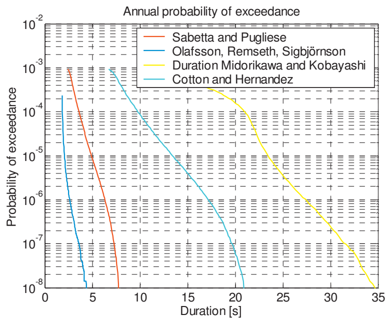
Arias intensity, stiff soil, Gutenberg Richter relationship, b = 1.3, a = 3.5,
Arias intensity probability of exceedance,
uniform hazard, radius 75 km (left) – fault 40 km shortest distance (right)



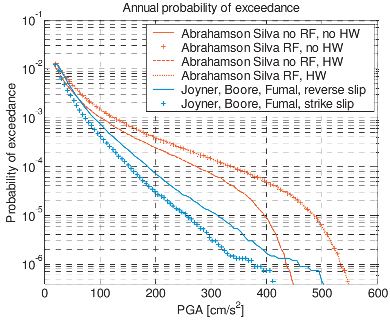


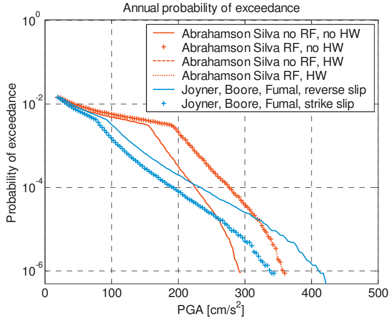
Duration, stiff soil, Gutenberg Richter relationship, b = 1.3, a = 3.5,
Duration probability of exceedance,
uniform hazard, radius 75 km (left) – fault 40 km shortest distance (right)



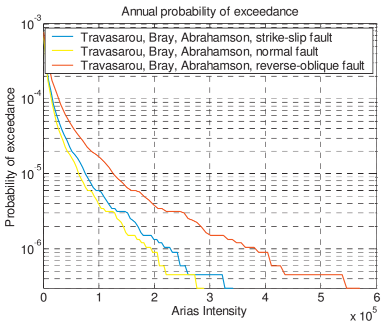


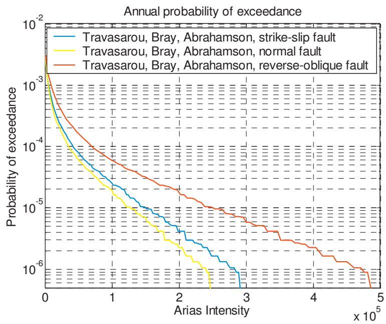
Changing fault parameters, stiff soil, Gutenberg Richter relationship, b = 1.3, a = 3.5,
PGA probability of exceedance,
uniform hazard, radius 75 km (left) – fault 10 km shortest distance (right)





Changing fault parameters,, stiff soil, Gutenberg Richter relationship, b = 1.3, a = 3.5,
Arias intensity probability of exceedance,
uniform hazard, radius 75 km (left) – fault 10 km shortest distance (right)





Appendix F

ANSYS batch files

Pushover analysis wall

```
finish
/clear
/config,nres,40000

!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!
!!!!!!!!!!!!!!!!!!!!!!!!INPUT OF VARIABLES!!!!!!!!!!!!!!!!!!!!!!!!!!!
!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!

!* Input of name
/FILNAME,obenfest,1,on

!* Input of material parameters

fric= 0.6             !* friction coefficient
mtens=0.15            !* tensile strength of mortar joints in N/mm2
mshea=0.2             !* shear strength of mortar joints in N/mm2
softa=1               !* Parameter for decrease of mortar strength
betam=0.8             !* beta coefficient for softening of mortar joints
compm=3.5             !* compressive strength of masonry in N/mm2
bshea=1.5             !* shear strength of bricks
softb=1               !* Parameter for decrease of brick strength
betab=0.4             !* beta coefficient for softening of bricks
para=0                !* parameter for proper solution commands
thick=1200            !* thickness of wall in mm
dens=2e-9             !* density in kg/(mm^3*1000)
emod=2000             !* Youngs modulus in N/mm2
nuxy=0.1              !* Poissons ration

!* Input of solution parameters

adamp=0.62            !*value for mass damping (low frequencies and rigid body motion)
bdamp=0.0003          !*value for stiffness damping (high frequencies)
iwert=50              !*number of steps to calculate
```

```
heightm=1

*if,heightm,EQ,1,then
height=3950
*elseif,heightm,eq,2,then
height=4600
*elseif,heightm,eq,3,then
height=5600
*elseif,heightm,eq,4,then
height=6300
*elseif,heightm,eq,5,then
height=7300
*elseif,heightm,eq,6,then
height=8900
*elseif,heightm,eq,7,then
height=10400
*elseif,heightm,eq,8,then
height=12600
*elseif,heightm,eq,9,then
height=14600
*elseif,heightm,eq,10,then
height=17700
*elseif,heightm,eq,10,then
*endif

!* Input of geometry

width=3000          !*width of tower
yforce=-50

!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!
!!!!!!!!!!!!!!!!!!!!!!!!PREP7    PROCESSOR!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!
!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!

/PREP7

ET,1,PLANE42
KEYOPT,1,1,0
KEYOPT,1,2,0
KEYOPT,1,3,3
KEYOPT,1,5,0
KEYOPT,1,6,0
R,1,thick,          !* thickness in mm
!*

NSVR,1,840          !* defines number of variables for user-programmable element options
```

```
MPTEMP,,,,,,,
MPTEMP,1,0
MPDATA,DENS,1,,dens          !* Density in kg/(mm^3*1000)
MPTEMP,,,,,,,
MPTEMP,1,0
MPDATA,EX,1,,emod  !* N/mm^2
MPDATA,PRXY,1,,nuxy

TB,USER,1,1,13,
TBTEMP,0
TBDATA,,1,fric,mtens,mshea,softa,Betam        !* Material data, 1=no rotation of element coordinate system,
                                              !* .5=friction coefficient, .2=tensile strength of mortar joints in N/mm^2
                                              !* .1=shear strength of mortar joints in N/mm^2, unknown parameter,.5=beta
                                              !* coefficient for softening of mortar
TBDATA,,compm,bshea,softb,betab,para,para  !* 3.5=compressive strength of masonry in N/mm^2, 1.5=shear strength of bricks
                                           !* inN/mm^2, 1= unknown parameter, .5= beta coefficient for softening of bricks
TBDATA,,para,,,,,                          !* threetimes zero for solution commands


zahl=iwert                 !*number of elements in the modell



!* Geometry

!* Tower lower keypoints
k,1,0,0
k,2,width,0
k,3,width,height
k,4,0,height

l,1,2
l,2,3
l,3,4
l,4,1

FLST,5,2,4,ORDE,2
FITEM,5,1
FITEM,5,3
CM,_Y,LINE
LSEL, , , ,P51X
CM,_Y1,LINE
```

```
CMSEL,,_Y
!*
LESIZE,_Y1, , ,7, , , , ,0
!*
FLST,5,2,4,ORDE,2
FITEM,5,2
FITEM,5,4
CM,_Y,LINE
LSEL, , , ,P51X
CM,_Y1,LINE
CMSEL,,_Y
!*
LESIZE,_Y1, , ,height/200, , , , ,0
!*


a,1,2,3,4
CM,_Y,AREA
ASEL, , , ,       1
CM,_Y1,AREA
CHKMSH,'AREA'
CMSEL,S,_Y
!*
MSHKEY,2
AMESH,_Y1
MSHKEY,0
!*
CMDELE,_Y
CMDELE,_Y1
CMDELE,_Y2

nsel,s,loc,y,0
cm,unten,node
allsel

nsel,s,loc,y,height
cm,oben,node
allsel


!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!
!!!!!!!!!!!!!!!!!!!!!!!!SOLUTION PROCESSOR!!!!!!!!!!!!!!!!!!!!!!!!
!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!

/SOLU                        !*Open solution processor

!************************Solution parameters
```

```
!***********************

ANTYPE,trans,new      !*Define new transient solution
!*

LUMPM,off             !*no lumped mass matrix formulation, element dependent instead
AUTOTS,off            !*automatic time stepping off

rescontrol,define,last,last,10      !*restart files written for every load step (per step appr. 9MB), max 10 files


NSUBST,10,200,5,off  !*20 substeps, 200 maximum, 5 minimum, do not use 20 as a start for determination of time step size
AUTOTS,on             !*automatic time stepping off
CUTCONTROL,DSPLIMIT,100000     !*time step cutback, incremental displacement limit
CUTCONTROL,PLSLIMIT,0.05       !*time step cutback, maximum equivalent plastic strain
KBC,0                 !*loads are ramped
LNSRCH,0              !*no line search in the Newton-Raphson iteration
NCNV,0,,,,       !*do not terminate analysis if solution does not converge, further input
NEQIT,200             !*maximum number of equilibrium iterations
PRED,off              !*no predictor activated in Newton-Raphson operation

eresx,no              !*copy integration point results to the nodes for alle elements (not necessary)
alphad,adamp          !*value for mass damping (low frequencies and rigid body motion)
betad,bdamp           !*value for stiffness damping (high frequencies)

!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!
!!!!!!!!!!!!!*Supports!!!!!!!!!!!!!!!!!!!!!!!!!!!!
!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!

d,unten,ux,0          !*Displacement at node 1 in x-direction is given by the product of dskal*...
d,unten,uy,0          !*Displacement at node 1 in z-direction is given by the product of dskal*...
d,oben,uy,0           !*Displacement at node 1 in z-direction is given by the product of dskal*...
f,oben,fy,-500

!***************************Input of time history
!***************************


!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!
timint,off,all                !*not including time effects
NSUBST,5,50,2,off
ACEL,0,9810,0                 !*acceleration in y-direction

time,0.001                    !*time at the end of load step
timint,off,all                !*not including time effects
```

```
solve

!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!

!*******************************Calculation of time history
!*******************************

*DO,i,1,iwert,1
NSUBST,10,200,2
neqit,500

d,oben,ux,i*0.1
d,oben,uy,0
d,unten,uy,0
d,unten,ux,p

ACEL,0,9810,0
save
time,(i)
solve
*ENDDO



finish
```

Probabilistic input- Sample (must be used with a batch file fitted to the parameters used here)

```
/pds

pdanl,all,txt,      !*input for loop file

!* Definition of statistical input data
pdvar,fric,log1,fric,fric*0.185   !* Random variable,paramter,distribution type, mean, standard deviation
pdvar,mtens,log1,mtens,mtens*0.35
pdvar,mshea,log1,mshea,mshea*0.30
pdvar,softa,unif,0.5,1.5          !* Uniform distribution lower and upper value
pdvar,betam,tria,0.3,0.8,1        !* Triangular distribution,lower limit, most likely value, upper limit
pdvar,compm,log1,compm,compm*0.19
pdvar,bshea,log1,bshea,bshea*0.30
pdvar,softb,unif,0.5,1.5
pdvar,betab,tria,0.3,0.4,1
pdvar,dens,gaus,dens,dens*0.075
pdvar,emod,gaus,emod,emod*0.12
pdvar,nuxy,log1,nuxy,nuxy*0.25

pdvar,xskal,log1,1,xskal*0.2
pdvar,yskal,log1,1,yskal*0.2
pdvar,adamp,unif,adamp*0.625,adamp*1.325
pdvar,bdamp,unif,bdamp*0.625,bdamp*1.325


!* Definition of output data

pdvar,MAXSTRESS,resp
pdvar,StressY,resp
pdvar,Stressx,resp
pdvar,SHEAR,resp
pdvar,DISX,resp
pdvar,DISY,resp
pdvar,SRAT,resp
pdvar,EPEQ,resp
pdvar,EPPL,resp

!* Definition of calculation method
pdmeth,mcs,lhs

!* Number of simulations
pdlhs,20,1,rand,,all,,,,init
!* Execution
pdexe,solumn,ser,10,del
```

Appendix G

Additional information macroelements

Element, cf. table 4-7 and table 4-13	Purposes	Additional information
1	Sensitivities and scatter for an scenario earthquakes; Sensitivities of input parameters, distributions of output parameters; Impact of input parameter distributions	Width: 3000 mm Height: 3950 mm Material parameters as shown in table 4-6 Load parameters, if needed, according to picture 3-23, Spudich et al. Changing material properties: 1x All lognormal distributions 1x All normal distributions 1x Elastic modulus, mean twice as much as table 4-6 1x Tensile strength, mean twice as much as table 4-6 1x Density, mean twice as much as table 4-6
2	Influence of the height/width ratio, shear wall	Material properties after table 4-6, mean value, load parameters not changed Width: 2000 mm Height: 1500 mm – 9500 mm
3	Sensitivities and scatter for triumphal arch	Geometry according to figure 4-6, material parameters according to table 4-6
4	Sensitivities and scatter for a barrel vault	Width: 7000 mm Arch rise: 1700 mm Thickness: 400 mm Material parameters according to table 4-6 Load parameters, if needed, according to picture 3-23, Spudich et al.
5	Sensitivities and scatter for an arch	Width: 5000 mm Arch rise: 1200 mm, Thickness: 400 mm Material parameters according to table 4-6 Load parameters, if needed, according to picture 3-23, Spudich et al.
6	Sensitivities and scatter for vault type 1, pointed cross vault	Width: 4000 mm Arch rise: 3000 mm Thickness: 400 mm Material parameters according to table 4-6 Load parameters, if needed, according to picture 3-23, Spudich et al.

7	Sensitivities and scatter for vault type 2, arched cross vault Impact of input parameter distributions	Width: 5000 mm Arch rise: 3000 mm Thickness: 400 mm Material parameters according to table 4-6 Load parameters, if needed, according to picture 3-23, Spudich et al. Changing material properties: 1x All lognormal distributions 1x All normal distributions 1x Elastic modulus, mean twice as much as table 4-6 1x Tensile strength, mean twice as much as table 4-6 1x Density, mean twice as much as table 4-6
8	Sensitivities and scatter for cross section of nave	Total width: 23000 mm Total height: 16800 mm Material parameters according to table 4-6
9	Sensitivities and scatter for cross section of tower	Total width: 6000 mm x 6000 mm Total height: 30000 mm Material parameters according to table 4-6
10	Sensitivities and scatter for out-of-plane, shear wall	Total width: 80 mm Total height: 6300 mm Material parameters according to table 4-6
11	Sensitivities and scatter for out-of-plane, shear wall with top anchoring	Total width: 80 mm Total height: 6300 mm Anchoring at 5000 mm height with spring element Material parameters according to table 4-6

Appendix H

Questionnaire

Università degli Studi di Firenze
Dipartimento di Ingegneria Civile
CRIACIV Marcel Urban
Via Santa Marta 3
50139 Firenze

Firenze, 14.05.2007

Analisi di rischio delle strutture storiche

Egregio Signore/Signora,

La nostra università partecipa assieme all'Università Tecnica di Braunschweig, Germania, ad un progetto di ricerca internazionale sul tema della valutazione del rischio sull'ambiente costruito. Nell'ambito di questo progetto stiamo portando avanti uno studio sul rischio sismico relativo alle chiese storiche. Questo studio è unico nel suo genere e i risultati del questionario allegato ci consentiranno di mettere in risalto l'importanza delle strutture storiche relativamente al loro capitale culturale, finanziario ed umano. Grazie alle sue competenze, la compilazione del questionario da parte sua sarebbe di grandissimo aiuto per noi. Le assicuriamo inoltre di mantenere uno stretto riserbo sui dati richiesti.

La compilazione del questionario non dovrebbe rubare più di 15 minuti del suo tempo ma contribuirebbe notevolmente al successo scientifico di questo studio, anche nel caso in cui non potesse rispondere a tutte le domande. Apprezzeremo ogni risposta sarà in grado di fornirci. Questa indagine è già stata realizzata con notevole successo in Germania e speriamo sinceramente che lei potrà aiutarci a migliorarne i risultati ed accrescerne il valore.

Una volta riempito, la preghiamo di rispedirci il questionario entro il 31 settembre 2005. Nel caso in cui avesse domande o desiderasse informazioni aggiuntive, non esiti a contattarci al seguente indirizzo di posta elettronica: *murban@dicea.unifi.it*.

I risultati generali di questa indagine dovrebbero essere pronti alla fine di agosto 2005. Se vuole, può darci il suo indirizzo di posta elettronica in modo da avvertirla non appena il resoconto sarà disponibile in rete.

La ringraziamo anticipatamente per la sua collaborazione e il suo apprezzato aiuto.

Ing. Marcel Urban

Allegati: Questionario

Technische Universität Carolo-Wilhelmina Braunschweig
Internationales Graduiertenkolleg 802
Marcel Urban
Beethovenstraße 51
38106 Braunschweig

Braunschweig, 21.04.2007

Risikobewertungen von historischen Bauwerken

Sehr geehrte Damen,
sehr geehrte Herren.

Derzeit arbeitet unsere Hochschule zusammen mit der Universität Florenz an einem Forschungsprojekt zur Risikoanalyse von Bauwerken unter extremen Belastungen. Innerhalb dieses Projekts wird an einer Studie zur Bestimmung des Erdbebenrisikos von historischen Kirchenbauten gearbeitet. Die Ergebnisse dieser Studie sollen es ermöglichen, die Bedeutung der historischen, kulturellen sowie sozial-religiösen Werte dieser Bauten gegenüber normalen Wohn- und Bürogebäuden zumindest grundlegend zu erfassen. Mit Ihrem Wissen können Sie zum Erfolg dieser Studie beitragen und uns mit der Beantwortung des Fragebogens sehr behilflich sein.

Das Ausfüllen der Fragen sollte nicht länger als 15 Minuten Ihrer Zeit in Anspruch nehmen. Auch wenn Sie nicht alle Fragen beantworten können oder möchten, so tragen Sie mit der Beantwortung von Teilfragen gleichermaßen zum wissenschaftlichen Erfolg der Studie bei. Selbstverständlich werden alle mit diesem Fragebogen erhobenen Daten anonym und streng vertraulich behandelt.

Bitte senden Sie Ihre Antwort mit dem beiliegenden Umschlag bis zum 30. Dezember 2005 zurück oder faxen Sie den Fragebogen an folgende Nummer *0531/391-4592*. Bei Fragen oder Informationen zu dieser Studie können Sie uns gerne unter *0531/391-3377* oder unter *m.urban@tu-braunschweig.de* kontaktieren.

Die Resultate dieser Studie werden voraussichtlich ab Juni 2006 verfügbar sein. Bei Interesse an den Ergebnissen vermerken Sie dies bitte mit Ihrer Email-Adresse am Ende des Fragebogens. Wir übersenden Ihnen dann einen kurzen Überblick über die Studienergebnisse. Wir danken Ihnen schon jetzt für Ihre Unterstützung.

Mit freundlichen Grüßen,

Marcel Urban

Anlagen: Fragebogen
Frankierter Rückumschlag

I. CARATTERISTICHE GENERALI

1. Nome della chiesa

2. Distanza dalle costruzioni adiacenti

- ☐ < 1 Metro
- ☐ < 10 Metri
- ☐ < 50 Metri
- ☐ più di 50 metri

3. Indica la posizione della chiesa

- ☐ Edificio isolato
- ☐ Paese
- ☐ Città
- ☐ Metropoli

4. Anno di Costruzione

5. Ampliamenti successivi

Anno	Elemento dell'edificio
	Cripta
	Abside
	Manutenzione dell'edificio

6. Contrassegna gli elementi naturali e antropici presenti in prossimità (~ 20m) della chiesa

- ☐ Ruscello
- ☐ Fiume
- ☐ Muri di sostegno
- ☐ Scarpata
- ☐ Altro
- ☐ Altro

7. In quale periodo storico è stata costruita la chiesa

- ☐ Pre-Romanico
- ☐ Romanico
- ☐ Gotico
- ☐ Rinascimento
- ☐ Barocco
- ☐ Altro

II. INFORMAZIONI SUL VALORE ARTISTICO DELLA CHIESA

1. Numero di opere d'arte rimovibili

a. Quadri

- ☐ < 5
- ☐ 5-10
- ☐ 10-20
- ☐ 20-50
- ☐ 50-100
- ☐ >100

b. Statue

☐ < 5	☐ 5-10	☐ 10-20
☐ 20-50	☐ 50-100	☐ >100

c. Altro

☐ < 5	☐ 5-10	☐ 10-20
☐ 20-50	☐ 50-100	☐ >100

d. Specifica il tipo di opere d’arte incluse in “Altro”

Tipo	Numero approssimato

2. Numero di elementi architettonici ed artistici non rimovibili

Descrizione	Numero approssimato	Anno di realizzazione
Coro		
Croce		
Vetrate decorate		
Altare principale		
Altri altari		
Affreschi in copertura		
Affreschi su pareti verticali		

3. Importanza della chiesa

☐ Di quartiere	☐ Locale (Città)	☐ Regionale
☐ Nazionale	☐ Europea	☐ Mondiale

4. Indica se la chiesa è vincolata dai Beni Culturali

☐ Si ☐ Non

5. Se si è risposto sì in 4.), specificare il tipo di vincolo

III. VALUTAZIONE DEL RISCHIO UMANO

1. Numero annuale di turisti

- ☐ < 10
- ☐ 10-50
- ☐ 50-100
- ☐ 100-500
- ☐ 500-1000
- ☐ 1000-5000
- ☐ 5000-10.000
- ☐ > 10.000

2. Provenienza dei turisti (se possibile)

Distanza	Percentuale
< 100 km	
100 km - 500km	
500 km - 1000 km	
> 1000 km	

3. Numero delle persone che vivono nella chiesa o nelle costruzioni adiacenti

- ☐ < 5
- ☐ 5-10
- ☐ 10-20
- ☐ 20-50
- ☐ 50-100
- ☐ 100-200
- ☐ 200-500
- ☐ più di 500

4. Numero massimo di persone contemporaneamente presenti nella chiesa

- ☐ < 30
- ☐ 30-50
- ☐ 50-70
- ☐ 70-150
- ☐ 150-300
- ☐ 300-500
- ☐ 500-1000
- ☐ più di 1000

5. Numero di volte all'anno in cui si è verificata la massima affluenza di persone

- ☐ 1
- ☐ 2
- ☐ 3
- ☐ 4
- ☐ 5
- ☐ 5-8
- ☐ 8-12
- ☐ più di 12

6. Numero medio di partecipanti alle funzioni religiose

- ☐ < 10
- ☐ 10-20
- ☐ 20-50
- ☐ 50-75
- ☐ 75-100
- ☐ 100-150
- ☐ 150-200
- ☐ più di 200

7. Dimensione della parrocchia

- ☐ < 1 km²
- ☐ 1-3 km²
- ☐ 3-5 km²
- ☐ 5-10 km²
- ☐ 10-15 km²
- ☐ 15-20 km²
- ☐ 20-50 km²
- ☐ più di 50 km²

8. Numero di persone appartenenti alla parrocchia

- ☐ < 30
- ☐ 30-50
- ☐ 50-100
- ☐ 100-300
- ☐ 300-500
- ☐ 500-1000
- ☐ 1000-3000
- ☐ più di 3000

9. Numero di funzioni religiose celebrate durante la settimana

Giorno	Numero delle Messe
Lunedì	
Martedì	
Mercoledì	
Giovedì	
Venerdì	
Sabato	
Domenica	

IV. BILANCIO ECONOMICO

1. Fondi spesa ogni anno per manutenzione o ristrutturazione

2. Interventi di manutenzione o ristrutturazione negli ultimi 20 anni. Se si:

Entitá della spesa

Frequenza dello stesso tipo di intervento

Descrizione degli interventi

3. Tipo ed entitá di finanziamnenti/entrate annuali della parrocchia

Beni Culturali

Sovvenzione statale

Biglietto di entrata (per chiese visitate da turisti)

Donazione

Affitto

Visita guidata

Eventi culturali

altro (si prega di specificare)

4. Numero di persone che partecipano alle attivitá della parrocchia

	Numero	salario mensile
Tempo pieno		
Part-time		
Studenti		
Volontari		

5. Ci sono spese a causa di:

Affitto

Tasse

Manutenzione ordinaria

6. Indicare se la chiesa è assicurata (si prega di specificare quali parti della chiesa)

☐ Si ☐ Non

V. VALUTAZIONE DELLA SICUREZZA STRUTURALLE

1. Ci sono stati danni a causa di

☐ Nessun danno ☐ Terremoto ☐ Vento

☐ Alluvione ☐ Altri eventi estremi ☐

2. Se ci sono fessurazioni visibile, indica dove si trovano

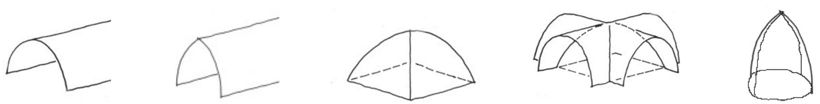
3. Se ci sono fessurazioni visibile, le crepe sono dovute a

☐ cedimento ☐ superamento della tensione ammissibile

☐ eventi estremi ☐ Altri(specificare):

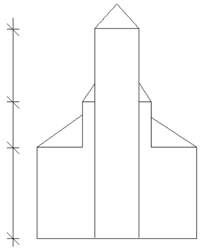
VI. GEOMETRIA

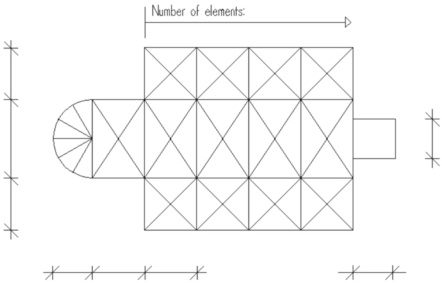
1. La forma delle cupole è simile a



☐	☐	☐	☐	☐

2. Dati geometrici





VII. VARIE

1. Accetto che le informazioni riportate nel questionario siano utilizzate per scopi scientifici

☐ Si ☐ Non

2. Saremmo lieti di ricevere ulteriori commenti e indicazioni su come migliorare questo questionario:

---- Grazie per la Sua collaborazione! ----

I. ALLGEMEINE CHARAKTERISTIKA

1. Name der Kirche: ______________________________

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2. Abstand zu benachbarten Gebäuden:

☐ < 1 Meter ☐ < 10 Meter ☐ < 50 Meter ☐ mehr als 50 Meter

3. Kreuzen Sie bitte an, in welchem Umfeld sich die Kirche befindet.

☐ allein stehendes Gebäude ☐ Dorf ☐ Stadt ☐ Großstadt

4. Erbaut im Jahr/in den Jahren: ______________________________

5. Erweiterung des Gebäudes in späteren Jahren:

Zeitpunkt	Teil des Gebäudes
	Krypta
	Apsis
	Betriebsgebäude

6. Welche der folgenden örtlichen Gegebenheiten sind vor Ort gegeben (bis etwa 20 Meter von der Kirche entfernt)?

☐ Bach ☐ Fluss ☐ Stützmauern

☐ Sumpf ☐ Abhänge ☐ …………………

7. Welcher Epoche / welchem Stil kann das Gebäude zugeordnet werden?

☐ Vor-Romanisch ☐ Romanisch ☐ Gotisch

☐ Renaissance ☐ Barock ☐ Andere

II. INFORMATIONEN ÜBER DIE KULTURELLEN GÜTER

1. Gegenstände von kultureller Bedeutung, die bewegt werden können:

Beschreibung	Ungefähre Anzahl	Alter in Jahren
Bilder		
Statuen		
Kruzifixe		

2. Gegenstände von kultureller Bedeutung, die nicht bewegt werden können:

Beschreibung	Ungefähre Anzahl	Alter in Jahren
Chor		
Kreuze		
Fenster von besonderer Bedeutung		
Hochaltar		
Nebenaltar		
Deckengemälde		
Wandgemälde		

3. Bekanntheitsgrad der Kirche und ihrer kulturellen Objekte:

☐ Dorf ☐ Stadt ☐ Region

☐ Bundesland ☐ Deutschland ☐ Weltbekannt

4. Steht die Kirche unter Denkmalschutz?

☐ Ja ☐ Nein

5. Wenn Sie Frage 4.) mit ja beantwortet haben, bitte geben Sie Träger und etwaige Besonderheiten an.

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III. INFORMATIONEN ÜBER PERSONENBEZOGENE RISIKEN

1. Wie groß ist die geschätzte Anzahl an Touristen im Jahr?

☐ weniger 10 ☐ 10-50 ☐ 50-100 ☐ 100-500

☐ 500-1.000 ☐ 1.000-5.000 ☐ 5.000-10.000 ☐ mehr als 10.000

2. Welche Entfernung legen die Touristen schätzungsweise zurück?

Distanz	Prozentzahl
Bis zu 100km	
Bis zu 500km	
Bis zu 1000km	
Mehr als 1000km	

3. Wie viele Personen wohnen in der Kirche oder in direkt angrenzenden Gebäuden?

☐ weniger 5 ☐ 5-10 ☐ 10-20 ☐ 20-50

☐ 50-100 ☐ 100-200 ☐ 200-500 ☐ mehr als 500

4. Was ist die maximale Personenanzahl, die sich in der Kirche aufhält?

☐ weniger 30 ☐ 30-50 ☐ 50-70 ☐ 70-150

☐ 150-300 ☐ 300-500 ☐ 500-1.000 ☐ mehr als 1.000

5. Wie oft wird diese Anzahl pro Jahr erreicht?

☐ 1	☐ 2	☐ 3	☐ 4
☐ 5	☐ 5-8	☐ 9-12	☐ mehr als zwölfmal

6. Wie groß ist die durchschnittliche Anzahl an Besuchern in einer Messe?

☐ weniger 20	☐ 20-50	☐ 50-75	☐ 75-100
☐ 100-150	☐ 150-250	☐ 250-400	☐ mehr als 400

7. Über welches Gebiet erstreckt sich die Gemeinde?

☐ weniger 1km²	☐ 3km²	☐ 3-5km²	☐ 5-10km²
☐ 10-15km²	☐ 15-20km²	☐ 20-50km²	☐ mehr als 50 km²

8. Wie viele Mitglieder hat die Gemeinde?

☐ weniger 100	☐ 100-300	☐ 300-500	☐ 500-1.000
☐ 1.000-2.000	☐ 2.000-3.000	☐ 3.000-4.000	☐ mehr als 4.000

9. Wie oft findet eine Messe statt?

Wochentag	Anzahl der Gottesdienste/Andachten
Montag	
Dienstag	
Mittwoch	
Donnerstag	
Freitag	
Samstag	
Sonntag	

IV. INFORMATIONEN ÜBER BAUAUSGABEN

1. Welcher Geldbetrag wird schätzungsweise pro Jahr für Renovierungen und Instandhaltung ausgegeben?

____

____

2. Fanden innerhalb der letzten 20 Jahre große Umbauten / Renovierungen statt?

a. Welche Summen wurden überschläglich ausgegeben: ______________

b. In welchen zeitlichen Abständen erwarten Sie erneut diese Kosten:

____

c. Was wurde renoviert: ______________________________

____

3. Sind Teile der Kirche versichert? Wenn ja, bitte geben Sie an, um welche Teile es sich handelt, bzw. was für Schäden die Versicherung abdeckt.

☐ Nein ______________________________

☐ Ja ______________________________

____

____

V. BEURTEILUNG DES TRAGWERKSZUSTANDS

1. Gab es Schäden, die einer der folgenden Belastungen zugeordnet werden können?

☐ Keine Schäden ☐ Erdbeben
☐ Wind ☐ Überschwemmungen
☐ Setzungen ☐ Andere (bitte angeben:) ______________

2. Gibt es sichtbare Risse im Mauerwerk? Wenn ja, bitte geben Sie grob an, wo diese liegen:

____

____

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____

____

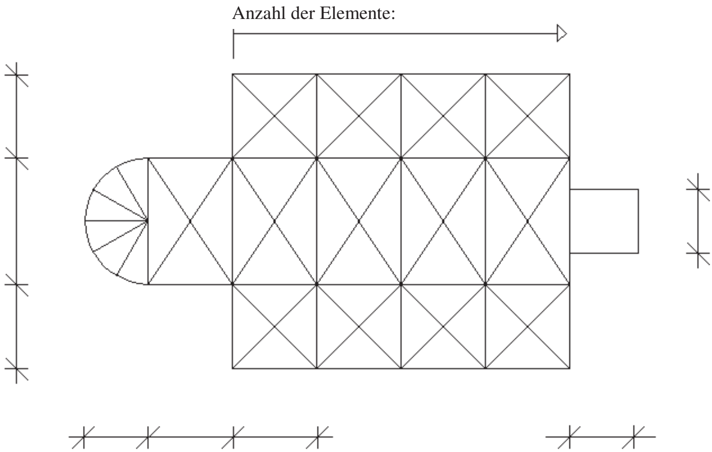
3. Wenn Sie Frage 2.) mit ja beantwortet haben, wurden die Risse durch eine der folgenden Punkte verursacht:

☐ Setzungen ☐ Überschreiten der maximalen Spannungen
☐ Extreme Einwirkungen ☐ ………………..

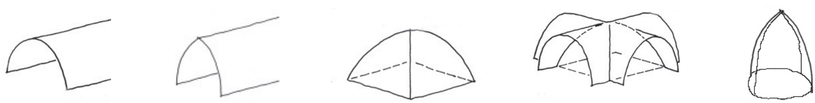
VI. INFORMATIONEN ÜBER DIE GEOMETRIE

1. Bitte kennzeichnen Sie in der folgenden Zeichnung die Lage und sofern bekannt die Maße von:

a. Anzahl der Schiffe
b. Anzahl sich wiederholender Querschnitte im Schiff
c. Die Lage des Turms
d. Die Lage und ggfls. Ausbildung des Chorraumes
e. Die Lage einer vorhandenen Krypta



2. Welche Form haben die Kuppeln in der Kirche?



☐ ☐ ☐ ☐ ☐

VII. VERSCHIEDENES

1. **Sofern Sie eine Website besitzen, auf der weitere Informationen zu erhalten sind, können Sie hier die Adresse angeben.**

2. **Wenn Sie an den Ergebnissen der Studie interessiert sind, können Sie hier Ihre Email-Adresse hinterlassen. Wir werden Sie dann bei Beendigung der Studie über die Resultate informieren.**

3. **An dieser Stelle haben Sie die Möglichkeit, uns Kommentare oder Anregungen zu diesem Fragebogen mitzuteilen.**

----------------Herzlichen Dank für Ihre Mitarbeit --------------------

Appendix I

Program to estimate the number of persons in a church

```
% MATLAB VERSION 6.5
% May be freely used at own risk
% Programm to derive artificial density distributions in a church
% Based on the Ph-Thesis "Earthquake risk assessment of historical
% structures"
% Author: Marcel Urban, Technische Universität Braunschweig, Germany, 2006
% Program modified and updated last time 06.04.2007

clear;
clc;
close all;
set(0,'defaultaxesfontsize',14);
set(0,'defaultlinelinewidth',2);


%______________INPUT__________________________

inhabitant=0;          % number of inhabitants
worker=0;              % number of workers
visitormean=5;         % mean number of visitors (tourists)
visitordev=0.1;        % standard deviation number of visitors (tourists)
sundaymean=70;         % mean number of participants religious events weekends
sundaydev=20;          % deviation number of participants religious events weekends
weekmean=30;           % mean number of participants religious events in the week
weekdev=8;             % deviation number of participants religious events win the week
numeventsb=5;          % number of week events !ONLY FULL NUMBERS,
MAXIMUM=15!
numeventsa=2;          % number of weekend events !ONLY FULL NUMBERS,
MAXIMUM=8!
highfmean=100;         % maximum number of persons in the church (due-to religious events)
mean
highfdev=20;           % maximum number of persons in the church (due-to religious events)
deviation
numevents=3;           % number of maximum events !ONLY FULL NUMBERS,
MAXIMUM=16!

%_______Normal Distribution / Year___________

visitor=normrnd(visitormean,visitordev,12,365);
sundaya=normrnd(sundaymean,sundaydev,numeventsa,365);
weeka=normrnd(weekmean,weekdev,1,365);
highf=normrnd(highfmean,highfdev,numevents,1);
```

```
%_______Distribution of visitors / year__________

factor=[0.411192958 0.524161273 1.016484579 1.642760784...
1.457549393 1.073403595 1.058736099 1.471774673 1.191608512...
0.992678878 0.590863525 0.568785728];

%___Creation of matrix hours and days / year_____
%________________24 hours________________________

matrix = zeros(24,365);
   matrix(1,:) = inhabitant;
   matrix(2,:) = inhabitant;
   matrix(3,:) = inhabitant;
   matrix(4,:) = inhabitant;
   matrix(5,:) = inhabitant;
   matrix(6,:) = inhabitant;
   matrix(7,:) = inhabitant+worker/2;
   matrix(8,:) = inhabitant+worker;
   matrix(9,:) = visitor(2,:)/4+worker;
   matrix(10,:) = visitor(3,:)/2+worker;
   matrix(11,:) = visitor(4,:)/4*3+worker;
   matrix(12,:) = visitor(5,:)+worker;
   matrix(13,:) = visitor(6,:)+worker;
   matrix(14,:) = visitor(7,:)+worker;
   matrix(15,:) = visitor(8,:)+worker;
   matrix(16,:) = visitor(9,:)+worker;
   matrix(17,:) = visitor(10,:)+worker;
   matrix(18,:) = visitor(11,:)/4+worker/2;
   matrix(19,:) = inhabitant+worker;
   matrix(20,:) = inhabitant;
   matrix(21,:) = inhabitant;
   matrix(22,:) = inhabitant;
   matrix(23,:) = inhabitant;
   matrix(24,:) = inhabitant;


%___________weekend approximately___________________
%__________more visitors during the weekend_________

z=1;
b=0;
z1=2;
z2=3;
z3=4;
z4=5;
z5=6;
for n=1:7:364                    % day
   c=8:1:19                      % hour
   z=z+7;
   b=b+7;
```

```
    matrix(c,z) =  matrix(c,z)*1.2;
    matrix(c,b) =  matrix(c,b)*1.3;
    matrix(c,z1) =  matrix(c,z1)*0.9;
    matrix(c,z2) =  matrix(c,z2)*0.9;
    matrix(c,z3) =  matrix(c,z3)*0.9;
    matrix(c,z4) =  matrix(c,z4)*0.9;
    matrix(c,z5) =  matrix(c,z5)*0.9;
end


%_______monthly distribution_____________
%___________January______________________
for n=1:31;                         % days
    c=8:1:19;                       % hours
    matrix(c,n) =  matrix(c,n)*factor(:,1);
end

%___________February_____________________
for n=32:59;
    c=8:1:19;
    matrix(c,n) =  matrix(c,n)*factor(:,2);
end

%___________March____________________
for n=60:90;
    c=8:1:19;
    matrix(c,n) =  matrix(c,n)*factor(:,3);
end

%___________April____________________
for n=91:120;
    c=8:1:19;
    matrix(c,n) =  matrix(c,n)*factor(:,4);
end

%___________May____________________
for n=121:151;
    c=8:1:19;
    matrix(c,n) =  matrix(c,n)*factor(:,5);
end

%___________June____________________
for n=152:181
    c=8:1:19;
    matrix(c,n) =  matrix(c,n)*factor(:,6);
end

%___________July___________________
for n=182:212
    c=8:1:19;
    matrix(c,n) =  matrix(c,n)*factor(:,7);
```

```
end

%__________August_________________
for n=213:243
   c=8:1:19;
   matrix(c,n) =  matrix(c,n)*factor(:,8);
end

%__________September_______________
for n=244:273
   c=8:1:19;
   matrix(c,n) =  matrix(c,n)*factor(:,9);
end

%_________October_________________
for n=274:304
   c=8:1:19;
   matrix(c,n) =  matrix(c,n)*factor(:,10);
end

%__________November________________
for n=305:334
   c=8:1:19;
   matrix(c,n) =  matrix(c,n)*factor(:,11);
end

%__________Dezember_______________
for n=335:365
   c=8:1:19;
   matrix(c,n) =  matrix(c,n)*factor(:,12);
end



%_______religious event weekend_______
z=1;
b=0;
for n=1:7:364
   q=1:53
   z=z+7;
   b=b+7;
   if numeventsa==1
      matrix(10,z) =  sundaya(1,z);
         elseif numeventsa==2
      matrix(10,z) =  sundaya(1,z);
      matrix(10,b) =  sundaya(2,b);
         elseif numeventsa==3
      matrix(10,z) =  sundaya(1,z);
      matrix(10,b) =  sundaya(2,b);
      matrix(18,z) =  sundaya(3,z);
         elseif numeventsa==4
```

```
      matrix(10,z) =  sundaya(1,z);
      matrix(10,b) =  sundaya(2,b);
      matrix(18,z) =  sundaya(3,z);
      matrix(18,b) =  sundaya(4,b);
        elseif numeventsa==5
      matrix(10,z) =  sundaya(1,z);
      matrix(10,b) =  sundaya(2,b);
      matrix(18,z) =  sundaya(3,z);
      matrix(18,b) =  sundaya(4,b);
      matrix(15,z) =  sundaya(5,z);
        elseif numeventsa==6
      matrix(10,z) =  sundaya(1,z);
      matrix(10,b) =  sundaya(2,b);
      matrix(18,z) =  sundaya(3,z);
      matrix(18,b) =  sundaya(4,b);
      matrix(15,z) =  sundaya(5,z);
      matrix(15,b) =  sundaya(6,b);
        elseif numeventsa==7
      matrix(10,z) =  sundaya(1,z);
      matrix(10,b) =  sundaya(2,b);
      matrix(18,z) =  sundaya(3,z);
      matrix(18,b) =  sundaya(4,b);
      matrix(15,z) =  sundaya(5,z);
      matrix(15,b) =  sundaya(6,b);
      matrix(12,z) =  sundaya(7,z);
        elseif numeventsa==8
      matrix(10,z) =  sundaya(1,z);
      matrix(10,b) =  sundaya(2,b);
      matrix(18,z) =  sundaya(3,z);
      matrix(18,b) =  sundaya(4,b);
      matrix(15,z) =  sundaya(5,z);
      matrix(15,b) =  sundaya(6,b);
      matrix(12,z) =  sundaya(7,z);
      matrix(12,z) =  sundaya(8,b);
  end

end



%_______religious event weekday_______
a=1;
b=-2;
c=-3;
d=-4;
e=-5;
if numeventsb==1
   for n=1:7:364 ;
   a=a+7;
   matrix(17,a) =  weeka(1,a);
   end
```

```
elseif numeventsb==2
    for n=1:7:364 ;
    a=a+7;
    b=b+7;
    matrix(17,a) =  weeka(1,a);
    matrix(17,b) =  weeka(1,b);
    end
elseif numeventsb==3
    for n=1:7:364 ;
    a=a+7;
    b=b+7;
    c=c+7;
    matrix(17,a) =  weeka(1,a);
    matrix(17,b) =  weeka(1,b);
    matrix(17,c) =  weeka(1,c);
    end
elseif numeventsb==4
    for n=1:7:364 ;
    a=a+7;
    b=b+7;
    c=c+7;
    d=d+7;
    matrix(17,a) =  weeka(1,a);
    matrix(17,b) =  weeka(1,b);
    matrix(17,c) =  weeka(1,c);
    matrix(17,d) =  weeka(1,d);
    end
elseif numeventsb==5
    for n=1:7:364 ;
    a=a+7;
    b=b+7;
    c=c+7;
    d=d+7;
    e=e+7;
    matrix(17,a) =  weeka(1,a);
    matrix(17,b) =  weeka(1,b);
    matrix(17,c) =  weeka(1,c);
    matrix(17,d) =  weeka(1,d);
    matrix(17,e) =  weeka(1,e);
    end
elseif numeventsb==6
    for n=1:7:364 ;
    a=a+7;
    b=b+7;
    c=c+7;
    d=d+7;
    e=e+7;
    matrix(17,a) =  weeka(1,a);
    matrix(8,a) =  weeka(1,a);
    matrix(17,b) =  weeka(1,b);
    matrix(17,c) =  weeka(1,c);
```

```
    matrix(17,d) =  weeka(1,d);
    matrix(17,e) =  weeka(1,e);
    end
elseif numeventsb==7
    for n=1:7:364 ;
    a=a+7;
    b=b+7;
    c=c+7;
    d=d+7;
    e=e+7;
    matrix(17,a) =  weeka(1,a);
    matrix(8,a) =  weeka(1,a);
    matrix(17,b) =  weeka(1,b);
    matrix(8,b) =  weeka(1,b);
    matrix(17,c) =  weeka(1,c);
    matrix(17,d) =  weeka(1,d);
    matrix(17,e) =  weeka(1,e);
    end
elseif numeventsb==8
    for n=1:7:364 ;
    a=a+7;
    b=b+7;
    c=c+7;
    d=d+7;
    e=e+7;
    matrix(17,a) =  weeka(1,a);
    matrix(8,a) =  weeka(1,a);
    matrix(17,b) =  weeka(1,b);
    matrix(8,b) =  weeka(1,b);
    matrix(17,c) =  weeka(1,c);
    matrix(8,c) =  weeka(1,c);
    matrix(17,d) =  weeka(1,d);
    matrix(17,e) =  weeka(1,e);
    end
elseif numeventsb==9
    for n=1:7:364 ;
    a=a+7;
    b=b+7;
    c=c+7;
    d=d+7;
    e=e+7;
    matrix(17,a) =  weeka(1,a);
    matrix(8,a) =  weeka(1,a);
    matrix(17,b) =  weeka(1,b);
    matrix(8,b) =  weeka(1,b);
    matrix(17,c) =  weeka(1,c);
    matrix(8,c) =  weeka(1,c);
    matrix(17,d) =  weeka(1,d);
    matrix(8,d) =  weeka(1,d);
    matrix(17,e) =  weeka(1,e);
    end
```

```
elseif numeventsb==10
    for n=1:7:364;
    a=a+7;
    b=b+7;
    c=c+7;
    d=d+7;
    e=e+7;
    matrix(17,a) =  weeka(1,a);
    matrix(8,a) =  weeka(1,a);
    matrix(17,b) =  weeka(1,b);
    matrix(8,b) =  weeka(1,b);
    matrix(17,c) =  weeka(1,c);
    matrix(8,c) =  weeka(1,c);
    matrix(17,d) =  weeka(1,d);
    matrix(8,d) =  weeka(1,d);
    matrix(17,e) =  weeka(1,e);
    matrix(8,e) =  weeka(1,e);
    end
elseif numeventsb==11
    for n=1:7:364;
    a=a+7;
    b=b+7;
    c=c+7;
    d=d+7;
    e=e+7;
    matrix(17,a) =  weeka(1,a);
    matrix(8,a) =  weeka(1,a);
    matrix(12,a) =  weeka(1,a);
    matrix(17,b) =  weeka(1,b);
    matrix(8,b) =  weeka(1,b);
    matrix(17,c) =  weeka(1,c);
    matrix(8,c) =  weeka(1,c);
    matrix(17,d) =  weeka(1,d);
    matrix(8,d) =  weeka(1,d);
    matrix(17,e) =  weeka(1,e);
    matrix(8,e) =  weeka(1,e);
    end
elseif numeventsb==12
    for n=1:7:364;
    a=a+7;
    b=b+7;
    c=c+7;
    d=d+7;
    e=e+7;
    matrix(17,a) =  weeka(1,a);
    matrix(8,a) =  weeka(1,a);
    matrix(12,a) =  weeka(1,a);
    matrix(17,b) =  weeka(1,b);
    matrix(8,b) =  weeka(1,b);
    matrix(12,b) =  weeka(1,b);
    matrix(17,c) =  weeka(1,c);
```

```
    matrix(8,c) =  weeka(1,c);
    matrix(17,d) =  weeka(1,d);
    matrix(8,d) =  weeka(1,d);
    matrix(17,e) =  weeka(1,e);
    matrix(8,e) =  weeka(1,e);
    end
elseif numeventsb==13
    for n=1:7:364;
    a=a+7;
    b=b+7;
    c=c+7;
    d=d+7;
    e=e+7;
    matrix(17,a) =  weeka(1,a);
    matrix(8,a) =  weeka(1,a);
    matrix(12,a) =  weeka(1,a);
    matrix(17,b) =  weeka(1,b);
    matrix(8,b) =  weeka(1,b);
    matrix(12,b) =  weeka(1,b);
    matrix(17,c) =  weeka(1,c);
    matrix(8,c) =  weeka(1,c);
    matrix(12,c) =  weeka(1,c);
    matrix(17,d) =  weeka(1,d);
    matrix(8,d) =  weeka(1,d);
    matrix(17,e) =  weeka(1,e);
    matrix(8,e) =  weeka(1,e);
    end
elseif numeventsb==14
    for n=1:7:364;
    a=a+7;
    b=b+7;
    c=c+7;
    d=d+7;
    e=e+7;
    matrix(17,a) =  weeka(1,a);
    matrix(8,a) =  weeka(1,a);
    matrix(12,a) =  weeka(1,a);
    matrix(17,b) =  weeka(1,b);
    matrix(8,b) =  weeka(1,b);
    matrix(12,b) =  weeka(1,b);
    matrix(17,c) =  weeka(1,c);
    matrix(8,c) =  weeka(1,c);
    matrix(12,c) =  weeka(1,c);
    matrix(17,d) =  weeka(1,d);
    matrix(8,d) =  weeka(1,d);
    matrix(12,d) =  weeka(1,d);
    matrix(17,e) =  weeka(1,e);
    matrix(8,e) =  weeka(1,e);
    end
elseif numeventsb==15
    for n=1:7:364;
```

```
    a=a+7;
    b=b+7;
    c=c+7;
    d=d+7;
    e=e+7;
    matrix(17,a) =  weeka(1,a);
    matrix(8,a) =  weeka(1,a);
    matrix(12,a) =  weeka(1,a);
    matrix(17,b) =  weeka(1,b);
    matrix(8,b) =  weeka(1,b);
    matrix(12,b) =  weeka(1,b);
    matrix(17,c) =  weeka(1,c);
    matrix(8,c) =  weeka(1,c);
    matrix(12,c) =  weeka(1,c);
    matrix(17,d) =  weeka(1,d);
    matrix(8,d) =  weeka(1,d);
    matrix(12,d) =  weeka(1,d);
    matrix(17,e) =  weeka(1,e);
    matrix(8,e) =  weeka(1,e);
    matrix(12,e) =  weeka(1,e);
    end
end

%_______religious event weekend_______
    if numevents==1
      matrix(18,350) =  highf(1,1);
        elseif numevents==2
      matrix(18,350) =  highf(1,1);
      matrix(18,351) =  highf(2,1);
        elseif numevents==3
      matrix(18,350) =  highf(1,1);
      matrix(18,51) =  highf(2,1);
      matrix(18,2) =  highf(3,1);
        elseif numevents==4
      matrix(18,350) =  highf(1,1);
      matrix(18,51) =  highf(2,1);
      matrix(18,2) =  highf(3,1);
      matrix(10,350) =  highf(4,1);
        elseif numevents==5
      matrix(18,350) =  highf(1,1);
      matrix(18,51) =  highf(2,1);
      matrix(18,2) =  highf(3,1);
      matrix(10,350) =  highf(4,1);
      matrix(10,351) =  highf(5,1);
        elseif numevents==6
      matrix(18,350) =  highf(1,1);
      matrix(18,51) =  highf(2,1);
      matrix(18,2) =  highf(3,1);
      matrix(10,350) =  highf(4,1);
      matrix(10,351) =  highf(5,1);
      matrix(10,352) =  highf(6,1);
```

```
  elseif numevents==7
matrix(18,350) =  highf(1,1);
matrix(18,51) =  highf(2,1);
matrix(18,2) =  highf(3,1);
matrix(10,350) =  highf(4,1);
matrix(10,351) =  highf(5,1);
matrix(10,352) =  highf(6,1);
matrix(23,350) =  highf(7,1);
  elseif numevents==8
matrix(18,350) =  highf(1,1);
matrix(18,51) =  highf(2,1);
matrix(18,2) =  highf(3,1);
matrix(10,350) =  highf(4,1);
matrix(10,351) =  highf(5,1);
matrix(10,352) =  highf(6,1);
matrix(23,350) =  highf(7,1);
matrix(18,120) =  highf(8,1);
  elseif numevents==9
matrix(18,350) =  highf(1,1);
matrix(18,51) =  highf(2,1);
matrix(18,2) =  highf(3,1);
matrix(10,350) =  highf(4,1);
matrix(10,351) =  highf(5,1);
matrix(10,352) =  highf(6,1);
matrix(23,350) =  highf(7,1);
matrix(18,120) =  highf(8,1);
matrix(18,121) =  highf(9,1);
  elseif numevents==10
matrix(18,350) =  highf(1,1);
matrix(18,51) =  highf(2,1);
matrix(18,2) =  highf(3,1);
matrix(10,350) =  highf(4,1);
matrix(10,351) =  highf(5,1);
matrix(10,352) =  highf(6,1);
matrix(23,350) =  highf(7,1);
matrix(18,120) =  highf(8,1);
matrix(18,121) =  highf(9,1);
matrix(18,122) =  highf(10,1);
  elseif numevents==11
matrix(18,350) =  highf(1,1);
matrix(18,51) =  highf(2,1);
matrix(18,2) =  highf(3,1);
matrix(10,350) =  highf(4,1);
matrix(10,351) =  highf(5,1);
matrix(10,352) =  highf(6,1);
matrix(23,350) =  highf(7,1);
matrix(18,120) =  highf(8,1);
matrix(18,121) =  highf(9,1);
matrix(18,122) =  highf(10,1);
matrix(10,120) =  highf(11,1);
  elseif numevents==12
```

```
matrix(18,350) =  highf(1,1);
matrix(18,51) =  highf(2,1);
matrix(18,2) =  highf(3,1);
matrix(10,350) =  highf(4,1);
matrix(10,351) =  highf(5,1);
matrix(10,352) =  highf(6,1);
matrix(23,350) =  highf(7,1);
matrix(18,120) =  highf(8,1);
matrix(18,121) =  highf(9,1);
matrix(18,122) =  highf(10,1);
matrix(10,120) =  highf(11,1);
matrix(10,121) =  highf(12,1);
  elseif numevents==13
matrix(18,350) =  highf(1,1);
matrix(18,51) =  highf(2,1);
matrix(18,2) =  highf(3,1);
matrix(10,350) =  highf(4,1);
matrix(10,351) =  highf(5,1);
matrix(10,352) =  highf(6,1);
matrix(23,350) =  highf(7,1);
matrix(18,120) =  highf(8,1);
matrix(18,121) =  highf(9,1);
matrix(18,122) =  highf(10,1);
matrix(10,120) =  highf(11,1);
matrix(10,121) =  highf(12,1);
matrix(10,122) =  highf(13,1);
  elseif numevents==14
matrix(18,350) =  highf(1,1);
matrix(18,51) =  highf(2,1);
matrix(18,2) =  highf(3,1);
matrix(10,350) =  highf(4,1);
matrix(10,351) =  highf(5,1);
matrix(10,352) =  highf(6,1);
matrix(23,350) =  highf(7,1);
matrix(18,120) =  highf(8,1);
matrix(18,121) =  highf(9,1);
matrix(18,122) =  highf(10,1);
matrix(10,120) =  highf(11,1);
matrix(10,121) =  highf(12,1);
matrix(10,122) =  highf(13,1);
matrix(10,222) =  highf(14,1);
  elseif numevents==15
matrix(18,350) =  highf(1,1);
matrix(18,51) =  highf(2,1);
matrix(18,2) =  highf(3,1);
matrix(10,350) =  highf(4,1);
matrix(10,351) =  highf(5,1);
matrix(10,352) =  highf(6,1);
matrix(23,350) =  highf(7,1);
matrix(18,120) =  highf(8,1);
matrix(18,121) =  highf(9,1);
```

```
      matrix(18,122) =  highf(10,1);
      matrix(10,120) =  highf(11,1);
      matrix(10,121) =  highf(12,1);
      matrix(10,122) =  highf(13,1);
      matrix(10,222) =  highf(14,1);
      matrix(23,200) =  highf(1,15);
        elseif numevents==16
      matrix(18,350) =  highf(1,1);
      matrix(18,51) =  highf(2,1);
      matrix(18,2) =  highf(3,1);
      matrix(10,350) =  highf(4,1);
      matrix(10,351) =  highf(5,1);
      matrix(10,352) =  highf(6,1);
      matrix(23,350) =  highf(7,1);
      matrix(18,120) =  highf(8,1);
      matrix(18,121) =  highf(9,1);
      matrix(18,122) =  highf(10,1);
      matrix(10,120) =  highf(11,1);
      matrix(10,121) =  highf(12,1);
      matrix(10,122) =  highf(13,1);
      matrix(10,222) =  highf(14,1);
      matrix(23,200) =  highf(15,1);
      matrix(23,202) =  highf(16,1);
  end


%_____________sort____________

matrixa = zeros(1,8760);

for n=1:365;
   matrixa(1,n) =  matrix(1,n);
end

for n=366:730;
   matrixa(1,n)=matrix(2,(n-365));
end

for n=731:1095;
   matrixa(1,n)=matrix(3,(n-730));
end

for n=1096:1460;
   matrixa(1,n)=matrix(4,(n-1095));
end


for n=1461:1825;
   matrixa(1,n)=matrix(5,(n-1460));
end
```

```
for n=1826:2190;
    matrixa(1,n)=matrix(6,(n-1825));
end


for n=2191:2555;
    matrixa(1,n)=matrix(7,(n-2190));
end


for n=2556:2920;
    matrixa(1,n)=matrix(8,(n-2555));
end


for n=2921:3285;
    matrixa(1,n)=matrix(9,(n-2920));
end


for n=3286:3650;
    matrixa(1,n)=matrix(10,(n-3285));
end


for n=3651:4015;
    matrixa(1,n)=matrix(11,(n-3650));
end


for n=4016:4380;
    matrixa(1,n)=matrix(12,(n-4015));
end


for n=4381:4745;
    matrixa(1,n)=matrix(13,(n-4380));
end


for n=4746:5110;
    matrixa(1,n)=matrix(14,(n-4745));
end


for n=5111:5475;
    matrixa(1,n)=matrix(15,(n-5110));
end
```

```
for n=5476:5840;
   matrixa(1,n)=matrix(16,(n-5475));
end


for n=5841:6205;
   matrixa(1,n)=matrix(17,(n-5840));
end


for n=6206:6570;
   matrixa(1,n)=matrix(18,(n-6205));
end


for n=6571:6935;
   matrixa(1,n)=matrix(19,(n-6570));
end

for n=6936:7300;
   matrixa(1,n)=matrix(20,(n-6935));
end

for n=7301:7665;
   matrixa(1,n)=matrix(21,(n-7300));
end

for n=7666:8030;
   matrixa(1,n)=matrix(22,(n-7665));
end

for n=8031:8395;
   matrixa(1,n)=matrix(23,(n-8030));
end

for n=8396:8760;
   matrixa(1,n)=matrix(24,(n-8395));
end



scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(1)
surfc(matrix);
title('Density Distribution of Person')
xlabel('Day');
ylabel('Hour');
zlabel('Number of Persons');
shading interp
```

```
scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(2)
mesh(matrix);
title('Density Distribution of Person')
xlabel('Day');
ylabel('Hour');
zlabel('Number of Persons');
shading interp

scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(3)
surface(matrix);
shading interp
title('Density Distribution of Person')
xlabel('Day');
ylabel('Hour');
zlabel('Number of Persons');
axis ij
axis([0 365 0 24 0 300])

scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(4)
surfl(matrix);
shading interp
colormap(gray);
title('Density Distribution of Person')
xlabel('Day');
ylabel('Hour');
zlabel('Number of Persons');
axis ij

scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(5)
hist(matrixa,20);
xlabel('Number of Persons')
ylabel('Absolute Frequency hourly basis')
title('Histogram Persons per Hour')

scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
```

```
figure(6)
hist(matrixa,50);
xlabel('Number of Persons')
ylabel('Absolute Frequency hourly basis')
title('Histogram Persons per Hour')

a=mean(matrixa)
b=std(matrixa)
c=max(matrixa)
d=min(matrixa)

hours=cumsum(matrixa);
total=max(hours);
hoursa=hist(matrixa,100);
lifes=total/8760
hoursb=cumsum(hoursa);
totala=max(hoursb);
perscdf=d:(c-d)/99:c;
cdf=1.*hoursb/totala;
cdfa=1-cdf;

scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(7)
plot(perscdf,cdf);
xlabel('Number of Persons')
ylabel('Probability')
title('CDF Number of Persons in the church')


scrsz = get(0,'ScreenSize');
figure('Position',[10 100 (scrsz(3)-10) (scrsz(4)-180)])
set(gca,'fontsize',16); %axis
figure(8)
semilogy(perscdf,cdfa);
xlabel('Number of persons')
ylabel('Probability')
title('Probability of Exceedance')
grid on
```

Appendix J

Curriculum vitae

Name:	Marcel Urban
Year of birth:	1976
Place of birth:	Wickede (Ruhr)
1985 – 1995	Walram Gymnasium Menden, Germany
1995 – 1997	Military Service, Germany
1997 – 2002	Student of Civil Engineering, Technische Universität Dresden, Germany Rensselaer Polytechnic Institute, Troy, NY, USA
2002	Research Assistant Technische Universität Dresden, Germany
2003 – 2005	Technische Universität Carolo-Wilhelmina zu Braunschweig, Institute for Steel Structures, Germany Università degli Studi di Firenze, Italy, Internationales Graduiertenkolleg 802 “Risk Management of Natural and Anthropogene Hazards on the Built Environment

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